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"An Investigation of the Stability of Bed Materials in a Stream of Water."†

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Notation.

l denotes the depth of a rectangular prism, or the side of a cubical block.
L ,, length of a rectangular prism or other object.
B ,, breadth of a rectangular prism or other object.
ρ' ,, density of a material.
ρ ,, density of water (assumed to be 62·4 lb. per cubic foot).
ρ_1 ,, quantity $(\rho' - \rho)$.
σ' ,, specific gravity of a material.
σ ,, specific gravity of water (assumed to be unity).
M ,, uniformity modulus of a material.
μ ,, coefficient of friction between a block and the bed of the channel.
v ,, velocity at a height y above the bed.
v_s ,, surface velocity.
v_{max} ,, maximum velocity.

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$\bar{v}$ denotes the mean velocity over the cross-section of the channel.
v_1 „ velocity halfway between the water-surface and the top of the object.
v_2 „ mean velocity in the centre of the flume between the water-surface and the top of the obstruction.
D „ total depth of water.
h „ depth of water above the obstruction.
m „ hydraulic mean depth.
d „ mean grain-size of a material.

Introduction.

Little quantitative information appears to be available as to the least size and weight of material which may safely be employed in the construction of embankments, causeways, or similar structures across a river, estuary, or tidal sound. The first object of the investigation described in this Paper was, therefore, to establish the magnitude of the current-velocities which cause movement of regular-shaped blocks placed either systematically in successive rows, ultimately forming an embankment, or laid in random fashion to form a mound. Secondly, the investigation was directed at finding the corresponding velocities if, instead of using, say, cubical concrete blocks, the work were carried out in flexible bolsters containing stone chippings. Finally, experiments were made on flat beds and mounds composed of chippings only.

The work was carried out in the Whitworth Engineering Laboratories at Manchester University, and special attention was devoted to the possibility of scale-effect in laboratory experiments of this kind. It is believed that sufficient has been done to enable the results to be applied with considerable confidence to full-size problems.

Theoretical Discussion.

The movement of a single block of material along the bed of a channel may be caused by

(i) the impact of the stream on the upstream face of the block ;
(ii) the reduction of pressure on the top and lee faces ;
(iii) the drag of the current, in particular along the top and sides of the block.

The first of these effects may be discussed in terms of the principle of momentum ; the other two are complicated by the influence of eddy-formation. But it is to be expected that each effect will be proportional approximately to the square of the current velocity.

Consider then the equilibrium of a single block, say a cube, resting on the horizontal bed of a stream. Its resistance to overturning is measured by a moment about the downstream lower edge of magnitude

$$\frac{(\rho' - \rho)l^4}{2},$$

where ρ' denotes the density of the cube, ρ the density of water, and l the length of the side of the cube.

Its resistance to sliding is a force of magnitude $\mu(\rho' - \rho)l^3$, where μ is the coefficient of friction between the cube and the bed.

Now let v denote the velocity of the current at a height y above the bed, and let it be assumed that the stream is broad in comparison with l, so that v may be taken to be constant across the width of the cube.

Then the normal impact of the current against the upstream face of the block produces a nominal overturning moment of

$$\frac{\rho l}{g}\int_0^l v^2 y \,.\, dy$$

and a nominal resultant horizontal force of

$$\frac{\rho l}{g}\int_0^l v^2 \,.\, dy.$$

Hence, neglecting any other forces which may tend to upset the block and assuming the entire forward momentum of the impinging stream to be destroyed, the cube will overturn if

$$\int_0^l v^2 y \,.\, dy > \frac{g(\rho' - \rho)l^3}{2\rho} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

or will slide if

$$\int_0^l v^2 \,.\, dy > \frac{\mu g(\rho' - \rho)l^2}{\rho} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

It is clear from these expressions that the condition of equilibrium is essentially dependent upon the manner in which the velocity is distributed through the depth of the stream. Certain possibilities will therefore be examined.

Let v_s denote the surface velocity; $v_{\max}$ the maximum velocity; and $\bar{v}$ the mean velocity.

Firstly, suppose that the velocity is uniform. Then $v_s = \bar{v} = v_{\max}$, and the condition for overturning becomes

$$v_s^2 > \frac{g(\rho' - \rho)}{\rho} \,.\, l \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

whilst that for sliding is

$$v_s^2 > \frac{\mu g(\rho' - \rho)}{\rho} \,.\, l \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

In this case, therefore, the velocity at which the cube will be disturbed is independent of the depth of water, is proportional to the square root of the dimension of the block and, on a plane surface, is in general smaller for sliding than for overturning.

Secondly, suppose that the velocity varies linearly from zero at the bed to v_s at the surface, the total depth of water being D:

then
$$\frac{v}{y} = \frac{v_s}{D}$$
$$\bar{v} = \frac{v_s}{2}$$
$$v_{\max} = v_s,$$

and the cube will overturn if

$$v_s^2 > 2g\left(\frac{\rho' - \rho}{\rho}\right)\left(\frac{D}{l}\right)^2 . l \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

and will slide if

$$v_s^2 > 3\mu g\left(\frac{\rho' - \rho}{\rho}\right)\left(\frac{D}{l}\right)^2 . l \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

These results indicate that the cube will overturn rather than slide if $\mu > 2/3$. Again, in this case of velocity-distribution, the velocity to produce movement is directly proportional to (D/l) and to $\sqrt{l}$.

Thirdly, let the velocity-distribution follow a parabolic law from zero at the bed to a maximum at the surface. Such a distribution might be approximately attained in a channel having a rough bed of uneven contour.

Here,
$$\frac{v}{v_s} = \sqrt{\frac{y}{D}},$$
$$v_{\max} = v_s$$
$$\bar{v} = \frac{2}{3}v_s.$$

For overturning,

$$v_s^2 > \frac{3}{2}g\left(\frac{\rho' - \rho}{\rho}\right)\left(\frac{D}{l}\right) . l \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

whilst for sliding,

$$v_s^2 > 2\mu g\left(\frac{\rho' - \rho}{\rho}\right)\left(\frac{D}{l}\right) . l \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

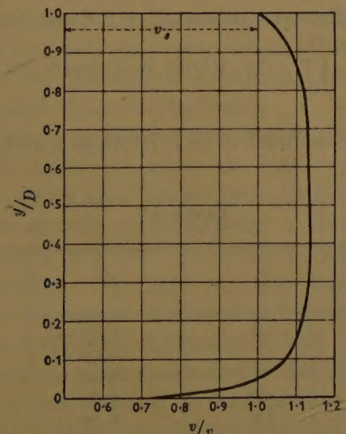
These results are of interest in indicating that with such a distribution, the velocity at which movement of a given block begins is proportional to the square root of the depth of water; in a stream of given depth it is independent of the size of block. If then the whole mechanical system of forces tending to produce movement could be expressed in this general manner, namely, $v^2 \propto \left(\frac{\rho' - \rho}{\rho}\right)\left(\frac{D}{l}\right) . l$, it would follow that, given such a parabolic velocity-distribution, the same size and weight of block would be moved in a model stream as in its full-scale prototype, provided that the velocity-scale were chosen to be proportional to the square root of the

depth. An analogous result has been found to be approximately true in many tidal model experiments where sand of the same order of size as in Nature has provided similarity of movement.

This discussion is intended only to convey a qualitative explanation of certain observed phenomena, since no account has been taken of the shape of the particle or of its interlocking with other grains. It does serve, however, to demonstrate that the phenomenon to which reference has been made is not so improbable as would appear at first sight. Further reference to the subject of tidal models will be made at a later stage.

In channels with nominally flat and smooth beds, the velocity near the bottom will be much higher than is represented by the case of parabolic distribution which has just been visualized: the actual velocity-distribution in the flumes utilized for the experiments described later has been measured and has been found to vary somewhat with different depths and average flows, but a mean of several curves has been drawn and is reproduced as *Fig. 1.*

Fig. 1.



This curve has been integrated arithmetically for the summations

$$\frac{\rho l}{g}\Sigma v^2 y \,.\, dy \text{ and } \frac{\rho l}{g}\Sigma v^2 \,.\, dy.$$

Let the surface velocity for overturning be given by

$$v_s = \alpha\sqrt{\frac{\rho' - \rho}{\rho}l} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

and that for sliding by

$$v_s = \beta\sqrt{\mu\frac{\rho' - \rho}{\rho}l} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Then the values of α and β appropriate to the curve of *Fig. 1* are as given in Table I :—

TABLE I.

$\frac{D}{l}$	$\frac{h}{l}$	α	β
1	0	5·06	5·14
2	1	5·08	5·19
5	4	5·25	5·46
10	9	5·55	5·81
15	14	5·81	6·09

Here D denotes the total depth and h the depth over the top of the cube ; when substituted in equations (9) and (10), these values of α and β will give velocities in feet per second if l is measured in feet. The analysis leading to these results assumes, however, that the forward momentum of the impinging stream is completely destroyed by the impact. This is not true, for the filaments near the edges of the face exposed to the current retain an appreciable forward momentum. Consequently, the real force of impact becomes $\frac{K\rho l}{g}\int_0^l v^2 \,.\, dy$, where K appears to have a value of the order of 0·70[1].

If the value of K is assumed to be 0·70, then α and β take the values set out in Table II.

TABLE II.

$\frac{D}{l}$	$\frac{h}{l}$	α	β
1	0	6·06	6·15
2	1	6·09	6·21
5	4	6·30	6·55
10	9	6·65	6·96
15	14	6·96	7·30

One other theoretical consideration deserves mention ; that is the possibility of the velocity-distribution curve varying with the depth of the channel and with the mean velocity in it. The general tendency is, in fact, for the filament of maximum velocity to approach more closely to the surface as the stream becomes shallower and its bed rougher.

From the standpoint of the force of impact on the upstream face, the smoother the bed the lower will be the mean velocity required to shift a block ; correspondingly there will be a marked tendency for the block to

[1] A. H. Gibson, "Hydraulics and its Applications," 3rd ed., p. 389. Constable 1925.

move first by sliding rather than by overturning. Even with a block having smooth faces, however, this tendency to slide first is negatived if the lee edge rests against some relatively small irregularity in the otherwise plane or smooth surface of the bed. This fact, together with the pronounced overturning moment of any suction on the top or lee face, explains why such a block frequently moves first by overturning even when the bed is so-called " smooth " or " plane."

Experimental Investigation.

The experiments were made in two concrete flumes, respectively 12 inches and 21 inches wide, through which water was circulated by a centrifugal pump. Velocities were measured by means of a calibrated pitometer coupled with a vertical differential gauge of the usual inverted U-tube type having air as the upper fluid. These velocities were measured in the centre of the flume, halfway between the free water-surface and the mid-point of the top of the block or other object under investigation. In a large number of cases, however, complete traverses of velocity were made in the central vertical line between the water-surface and the object, and the mean velocity of flow over the cross-section at the middle of the obstacle was also determined by the aid of a venturi meter incorporated in the pipe-line which supplied water to the flume.

Preliminary observations showed that the way in which a cubical block first moved depended upon just where the block was placed on the bed of the flume, and upon which side was in contact with the bed. Thus, sometimes it would slide, remain stationary for a short time, slide again for some distance, overturn, and roll. On other occasions its first movement would be by overturning. Friction tests were made by dragging cement blocks along the bed by means of a cord and spring balance ; these tests indicated an average coefficient of friction of 0·62.

Having regard to these phenomena and to the theoretical considerations already discussed, it was decided to standardize the conditions of the experiments by fixing a sheet-iron plate 0·094 inch thick across the width of the flume. The various objects used were then placed with one edge abutting this plate, and the flow of water was increased in gradual and regular steps until movement was observed. The pitometer and depth readings were taken immediately before this movement occurred.

The object of using flumes of different widths was two-fold : firstly, to determine any scale effect due to width, and secondly, to enable the higher velocities, as demanded by the larger objects, to be obtained with the pump available.

For the purpose of checking observations, many of the tests were undertaken several times : these justify the statement that the velocities quoted as being those for initial movement are consistent within about ± 10 per cent. ; often they lie within a very much narrower band. Nor is any

greater accuracy likely to be attained in such tests, considering the difference which might be expected from slight irregularities in the shape of the object tried, and consequently its varying behaviour according to which face is used to receive the current. There is, moreover, the well-known difficulty, in this kind of experiment, of estimating the precise moment of initial movement.

The following notation is adopted in presenting the results :

v_1 denotes the velocity measured midway between the water-surface and the top of the object ;

v_2 denotes the mean velocity, in the centre of the flume, between the surface and the top of the object ;

v_{max} denotes the maximum velocity between the water-surface and the top of the object ;

v_s denotes the velocity in the water-surface above the middle of the object ;

v denotes the mean velocity over a cross-section of the flume taken at the middle of the object.

Series 1.—Tests on single blocks placed in the centre of the flume.

For this series, blocks were cast in cement mortar ; two batches were made from different mixes, and after immersion in water the densities were 150·6 lb. and 130·4 lb. per cubic feet, respectively. These values are used as the effective values, so that $\frac{\rho' - \rho}{\rho}$ becomes

$$\frac{150{\cdot}6 - 62{\cdot}4}{62{\cdot}4} \text{ and } \frac{130{\cdot}4 - 62{\cdot}4}{62{\cdot}4},$$

or 1·41 and 1·09, respectively.

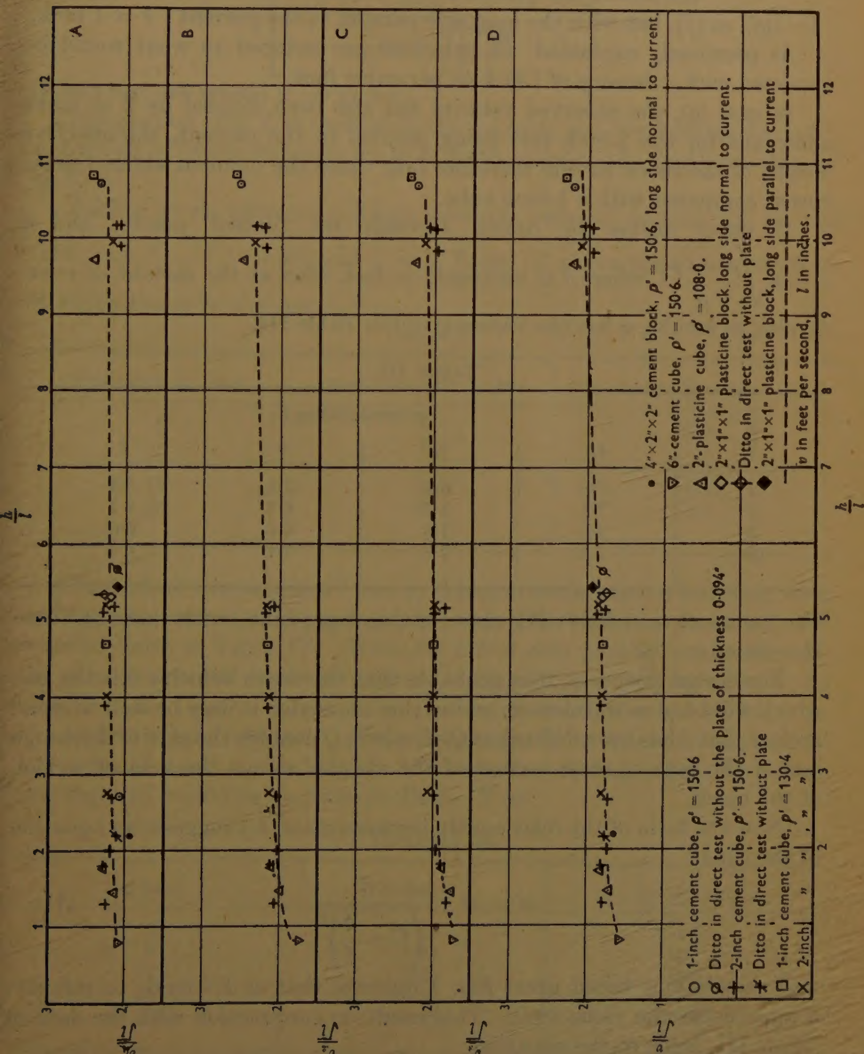
Blocks were also made of plasticine, for which $\frac{\rho' - \rho}{\rho}$ was found to be 0·733.

Observed velocities have, for comparative purposes, been reduced to what would be expected for an effective density-ratio of 1·09. Thus, the velocities for the heavier blocks have been multiplied by $\sqrt{\frac{1{\cdot}09}{1{\cdot}41}} = 0{\cdot}88$, and those for the plasticine by $\sqrt{\frac{1{\cdot}09}{0{\cdot}733}} = 1{\cdot}22$.

It is further desirable to adjust the results for the effect of the plate (0·094 inch thick). Calculations based on the curve of *Fig. 1* indicate that the velocities required to overturn the blocks are 10 per cent. higher when this plate is used with a block 1 inch deep than if no plate were used. The corresponding percentages for blocks 2 inches and 6 inches deep are

5 and 1·5, respectively. Observed velocities have accordingly been reduced by the allowance of these adjusting percentages.

The observations are summarized in *Figs. 2a, b, c, and d*; the ordinates used being $\frac{v_1}{\sqrt{l}}$, $\frac{v_2}{\sqrt{l}}$, $\frac{v_3}{\sqrt{l}}$, and $\frac{\bar{v}}{\sqrt{l}}$, respectively. Each v is measured in feet per second; l denotes the depth of the block normal to the bed of the flume, in inches. The abscissas represent the ratio h/l, h denoting the depth of water, in inches, from the top of the block.



Reference to *Figs. 2* will show that in this series the blocks used were :

(*a*) cement cubes, $l = 1$ inch,

(*b*) „ „ $l = 2$ inches,

(*c*) a cement block 4 inches by 2 inches by 2 inches, with its long side normal to the current ; $l = 2$ inches,

(*d*) a cement cube, $l = 6$ inches,

(*e*) a plasticine cube, $l = 2$ inches,

(*f*) a plasticine block, 2 inches by 1 inch by 1 inch, with its long side normal to the current ; $l = 1$ inch,

(*g*) as (*f*), but with the long side parallel to the current ; $l = 1$ inch.

As previously explained, all velocities are reduced to what would be expected with a density of 130·4 lb. per cubic foot.

In case (*g*), the observed velocity has also been divided by 2 to make allowance for the 2-inch side being parallel to the current, the effective weight of the block having therefore four times the moment about the lee corner compared with a 1-inch cube.

If mean curves be drawn through the plotted points, and if $v = \alpha\sqrt{\frac{\rho' - \rho}{\rho}l}$, where l is measured in feet, then at the instant of overturning of a cube, α has the values given in Table III.

TABLE III.

$\frac{h}{l}$	α corresponding to v_1	v_2	v_3	$\bar{v}$
1	7·0	6·2	5·8	5·3
2	7·1	6·7	6·3	5·7
5	7·3	7·3	6·6	6·0
10	7·5	7·8	7·1	6·9

The values of v_s in Table III agree, within 6 per cent., with those of Table II, column α.

For design purposes, it is probable that the mean velocity $\bar{v}$ is the one which will be of most interest, and in this connexion it may be well to make it clear that $\bar{v}$ has been defined as Q/A, where Q denotes the rate of discharge and A the area of cross-section of the channel minus the area of section of the block.

Now the form of the relationship between α and h/l suggests an equation of the type

$$\alpha = A - \frac{B}{\left(C + \frac{h}{l}\right)^n} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

and calculations based upon *Fig. 1* indicate that as h/l tends to infinity, α approaches the value 10·0. This result, in conjunction with the data of Table III, leads to the equation

$$\alpha = 10{\cdot}0 - \frac{5300}{\left(32 + \frac{h}{l}\right)^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (12)$$

This expression, in fact, satisfies the value of $\bar{v}$ implied in Table III within 5 per cent.

If then a block in the form of a rectangular prism of length L and depth l be considered, the mean velocity at which it will overturn is given by

$$\bar{v} = \frac{L}{l}\left[10{\cdot}0 - \frac{5300}{\left(32 + \frac{h}{l}\right)^2}\right]\sqrt{\frac{\rho' - \rho}{\rho}l} \quad . \quad . \quad . \quad . \quad . \qquad (13)$$

($\bar{v}$ is in feet per second if l is in feet).

In equation (13), L is parallel to the current, and the formula holds for any width of block certainly up to a width of twice the depth and for L/l up to 2.

Taking as an example a 3-foot concrete cube of density 150 lb. per cubic foot, $\frac{\rho' - \rho}{\rho} = 1{\cdot}40$, and $\frac{L}{l} = 1$, Table IV shows the mean velocities at which the cube will move.

TABLE IV.

Total depth D: feet.	Depth over cube, h: feet.	$\bar{v}$: feet per second.
6	3	10·5
9	6	11·1
12	9	11·6
18	15	12·5
33	30	14·3

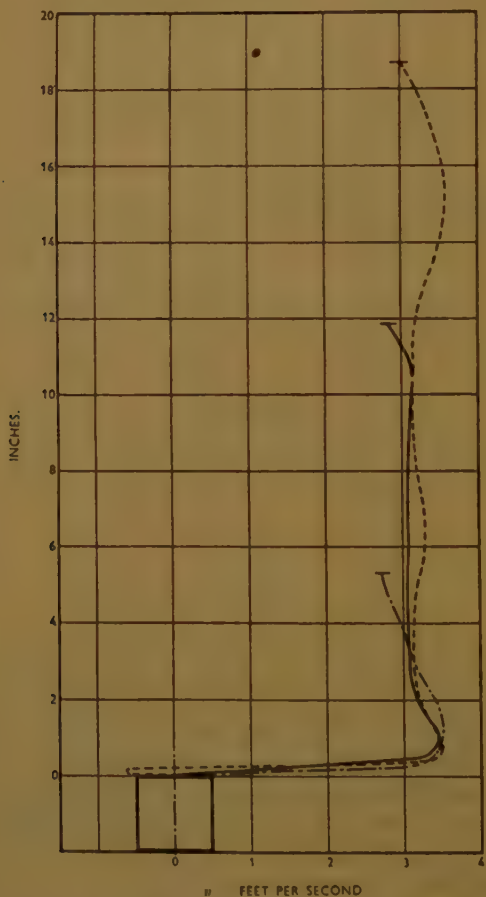
If the block were a prism 3 feet by 3 feet in section and 6 feet long, with its long side normal to the current, the velocities would be practically the same as those in Table IV. With the 6-foot side parallel to the current, the velocities would be twice as great. This block would have twice the weight of the original 3-foot cube, that is, its volume would be 54 cubic feet and its weight 3·62 tons. The same weight of material made in cubical form, would mean a cube of side 3·77 feet, for moving which the velocities required would be as given in Table IV (*a*):

TABLE IV (*a*).

D: feet.	h: feet.	$\bar{v}$: feet per second.
6	2·23	11·5
9	5·23	12·1
12	8·23	12·6
18	14·23	13·4
33	29·23	15·3

Fig. 3 shows velocity-curves observed at the centre of the flume, that is, over the middle of a cube. These curves are for three different values of h, namely, 5·3 inches, 11·85 inches, and 18·7 inches; they are all based upon observations made just before the movement of a 2-inch cube of wet

Fig. 3.



VELOCITY CURVES AT THE CENTRE OF THE FLUME.

density 130·4 lb. per cubic foot. A remarkable feature is that the maximum velocity is constant, within experimental limits, at 3·50 feet per second.

Suppose now that such a cube is subjected to a velocity-distribution against its upstream face, as indicated in *Fig. 4*. If the whole of the forward momentum of the impinging stream be assumed to be destroyed, the value of v' required to overturn the block about X is 3·42 feet per

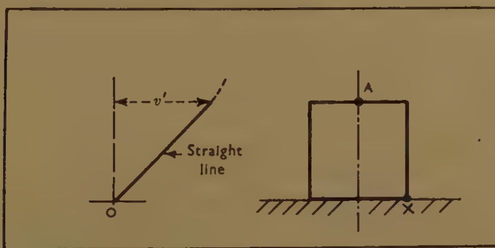
second. This is nearly the same as the maximum observed velocity between A and the water-surface, as quoted above.

With a 1-inch cube, v' would have to be 2·42 feet per second; two curves taken showed the maximum velocity to be 2·30 feet and 2·54 feet per second respectively.

Again, with a 6-inch cube, v' would be 5·91 feet per second; a curve taken with a 6-inch cube and with $h = 5$ inches showed a maximum velocity of 5·95 feet per second.

It appears, therefore, that a simple way of estimating the maximum

Fig. 4.



HYPOTHETICAL VELOCITY-DISTRIBUTION AGAINST UPSTREAM FACE OF A CUBE.

velocity which may be sustained between the top of a cube and the water-surface is to calculate it as equal to $\sqrt{2g\frac{\rho' - \rho}{\rho}l}$ feet per second, where l is in feet.

Series 2.—The investigation was next extended to cover the possibility of constructing, in stages, an embankment or mound of cubical blocks. Accordingly, the following arrangements of blocks, stretching across the full width of the flume, were tried:

Case (*a*)  A single row;
„ (*b*)  two rows;
„ (*c*)  three rows;
„ (*d*)  four rows;
„ (*e*) 
„ (*f*) 
„ (*g*) 
„ (*h*) 
„ (*i*) A pell-mell wall of cubes, dropped at random, to form an embankment of crest-height approximately the same as in case (*g*).

In carrying out these tests, any small gap left between the end of the wall and the side of the flume was filled with plasticine or other material pressed lightly into position.

In plotting the results, for comparative purposes the measured velocities have again been adjusted to what would be expected for an effective density-ratio, $\frac{\rho' - \rho}{\rho}$, of 1·09. Allowance for the plate 0·094 inch in thickness has also been made in case (*a*), but not otherwise, because with more than one row, it was frequently found that the first movement occurred by one or more blocks being lifted from the upstream rows right over the remainder of the cubes. In cases (*e*) and (*g*), it was generally observed that the top row first shifted into a slightly arched line, convex downstream, before one of its component blocks overturned.

Again, let $v = \alpha\sqrt{\frac{\rho' - \rho}{\rho} . l}$, where l denotes the side of the cube, in feet. From the various plotted results, it has been found that α has values as set out in Tables V a, b, c, d, e, f, g, h, i :

TABLE V (a).—CASE (*a*), SINGLE ROW.

$\frac{h}{l}$	Corresponding to v_1	v_2	v_s	$\bar{v}$
1	4·8	3·8	3·6	3·1
2	5·6	4·6	4·3	4·0
5	6·8	6·1	5·3	5·0
10	7·6	7·4	6·3	6·0

TABLE V (b).—CASE (*b*), TWO ROWS.

$\frac{h}{l}$	v_1	v_2	v_s	$\bar{v}$
1	6·6	5·6	5·8	5·0
2	7·1	6·5	6·5	5·3
5	8·1	7·8	7·4	6·3
10	9·0	8·8	8·3	7·6

TABLE V (c).—CASE (*c*), THREE ROWS.

$\frac{h}{l}$	v_1	v_2	v_s	$\bar{v}$
1	8·1	6·8	6·5	6·6
2	9·0	7·6	7·3	6·9
5	9·9	9·1	8·4	7·8
10	10·3	10·6	9·6	8·3

TABLE V (d).—CASE (*d*), FOUR ROWS.

$\frac{h}{l}$	v_1	v_2	v_s	$\bar{v}$
1	7·9	6·3	6·6	6·8
2	8·8	7·3	7·4	7·4
5	9·9	9·1	8·8	8·1
10	10·4	10·8	10·0	8·6

TABLE V (e).—CASE (*e*).

$\frac{h}{l}$	v_1	v_2	v_s	$\bar{v}$
1	4·6	4·5	3·3	3·2
2	5·6	5·0	4·0	3·9
5	6·6	6·2	5·1	5·0
10	7·1	7·1	6·0	6·3

TABLE V (f).—CASE (*f*).

$\frac{h}{l}$	v_1	v_2	v_s	$\bar{v}$
1	6·5	4·8	4·6	4·8
2	7·1	5·8	5·5	5·4
5	8·3	7·5	6·8	6·5
10	8·9	9·1	8·1	7·3

TABLE V (g).—CASE (*g*).

$\frac{h}{l}$	v_1	v_2	v_s	$\bar{v}$
1	3·8	3·1	3·0	2·2
2	4·6	3·8	3·5	3·0
5	5·8	5·1	4·5	4·6
10	6·6	6·6	5·3	6·3

TABLE V (h).—CASE (*h*).

$\frac{h}{l}$	v_1	v_2	v_s	$\bar{v}$
1	6·8	5·0	6·0	5·3
2	7·8	6·0	6·6	6·0
5	9·1	7·9	7·6	7·3
10	9·9	9·6	8·3	8·3

TABLE V (i).—CASE (*i*).—PELL-MELL MOUND.

$\frac{h}{l}$	v_1		v_2		v_3		$\bar{v}$	
	×	+	×	+	×	+	×	+
1	3·0	5·6	2·7	6·2	2·3	5·0	2·5	5·6
2	3·6	6·8	3·4	7·0	3·0	6·0	3·1	6·3
5	5·0	8·6	5·0	8·3	4·3	7·4	4·6	7·4
10	6·0	10·3	6·6	9·3	5·6	8·8	6·1	8·6

× : One or two cubes moved.
+ : Many cubes moved ; mound flattened to about two-thirds of its original height.

Confining attention to the mean velocity, $\bar{v}$, it is of interest to compare its values with those for a solitary cube. Table VI shows the velocities ($\bar{v}$) relative to those of Table III.

TABLE VI.—$\bar{v}$ RELATIVE TO $\bar{v}$ FOR A SINGLE CUBE.

$\frac{h}{l}$	*a*	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*	
									×	+
1	0·58	0·94	1·25	1·28	0·60	0·91	0·41	1·00	0·47	1·06
2	0·70	0·93	1·21	1·30	0·68	0·95	0·52	1·05	0·55	1·10
5	0·83	1·05	1·30	1·35	0·83	1·08	0·77	1·22	0·77	1·23
10	0·87	1·10	1·20	1·25	0·91	1·06	0·91	1·20	0·88	1·25

Several interesting conclusions emerge from these results :

(i) There is a definite tendency for the velocity with a single row resting either directly on the bed, or on other layers of cubes, to become the same as for a solitary cube, if the depth of water is very great.

(ii) If $\frac{h}{l} = 5$ or more, the velocity required to disturb a single row is sensibly identical whether the layer rests on the bed or on other blocks.

(iii) Two rows are disturbed by approximately the same $\bar{v}$ whether they rest on the bed or on a lower layer of blocks.

(iv) The same conclusion as (iii) holds for three rows if $\frac{h}{l} = 5$ or more.

(v) The $\bar{v}$ required to displace several blocks of the pell-mell arrangement is always of the same order as that for initial movement of case (*h*), where the height is about the same as the random mound after its several blocks have been shifted.

(vi) With one tier of successive rows of cubes (cases *a*, *b*, *c*, *d*), the stability approaches a limiting value. Thus, two rows have the same order of stability as a single isolated cube; three rows will withstand about 25 per cent. higher velocities than two rows, but four rows require only a further 5 per cent. increase.

(vii) Except with a relatively long flat bed of a single thickness of cubes, the effect of increasing the h/l ratio from, say, 1 to 10, is far more pronounced than in the case of an isolated cube: but in all cases the indications are that the effect of water-depth becomes comparatively small for values of h/l exceeding 10.

To provide a direct basis of comparison with Table IV, consider a mound to be formed by dumping 3-foot concrete cubes in random manner. Table VII gives the velocities required for initial disturbance of such a mound.

TABLE VII.

h, depth over crest of mound : feet.	3	6	15	30
$\bar{v}$: feet per second	4·9	6·1	9·6	12·6

The maximum velocities observed between the crests of these various obstructions and the water-surface are no longer constant, as in the example of a solitary cube; they now bear an approximately constant ratio, 1·35 : 1, to the mean velocities, $\bar{v}$.

Series 1A. Tests on flexible bolsters.

A possible way of constructing an underwater embankment is by the use of broken stone enclosed in wire cages. To investigate the stability of this type of construction, experiments were made on bolsters consisting of thin cotton bags filled with granite chippings. The true specific gravity of the chippings was 2·64; their average weight per piece was 0·00316 lb. Average dimensions of the pieces were found to be:

Maximum length of piece . . .	0·80 inch
Maximum breadth of piece . . .	0·49 ,,
Maximum thickness of piece . . .	0·29 ,,

The bolsters made in this way had a mean outside diameter of 2·1 inches and a length of 3·54 inches; the weight of each bolster was 0·571 lb. (dry), equivalent to a density of 80·3 lb. per cubic foot. Adopting this figure for ρ', $\dfrac{\rho'-\rho}{\rho}$ becomes 0·288. The dry weight of the bolster is the same (sensibly) as that of a 2-inch cement cube having a wet density of

130·4 lb. per cubic foot. In the discussion which follows, l is taken as the mean outside diameter of 2·1 inches.

In all the experiments, the bolsters were placed with their long dimension normal to the current, except in the case of a pell-mell mound, when they were dropped from just clear of the water-surface and allowed to settle in random fashion.

Considering first the results obtained with a single bolster placed in the centre of the flume, Table VIII shows the ratios of the various velocities necessary to overturn it to those for a 2-inch cube of the same dry weight.

TABLE VIII.

$\frac{h}{l}$	v_1	v_2	v_s	$\bar{v}$	v_{max}	Averages.
1	0·74	0·79	0·78	0·66	0·62	0·72
2	0·72	0·73	0·72	0·64	0·64	0·69
5	0·71	0·68	0·69	0·66	0·67	0·68
10	0·70	0·66	0·64	0·68	0·71	0·68

The mean velocities, $\bar{v}$, at which movement takes place are therefore only two-thirds of those with the cube. This means that a 2-inch cube weighing (dry) about 0·70 as much as the bolster would move under the same velocity. Similarly, a rectangular block 2 inches square in section and having the same length : depth ratio of 3·54 : 2·1, would move at about the same velocity as the bolster, if its weight were $\frac{3 \cdot 54}{2 \cdot 1} \times 0 \cdot 70$, or 1·18 times the weight of the bolster.

Now when the bolster was wet and lying on the bottom of the flume, it was found to have a depth of 1·8 inch, and its mean width would be therefore about $\frac{(2 \cdot 1)^2}{1 \cdot 8}$, or 2·45 inches. It is of interest to compare its behaviour with that of a solid rectangular prism, 1·8 inch deep, 2·45 inches wide, and 3·54 inches long, the longest dimension being normal to the stream. Assuming such a block to have a density, ρ', of 80·3 lb. per cubic foot, that is, the same as the dry density of the bolster, Table IX gives the velocities for overturning :

TABLE IX.

Total depth, D, of water : inches.	$\bar{v}$: feet per second.	
	Bolster.	Block.
3·9	1·44	1·46
6·0	1·48	1·55
12·3	1·73	1·78
22·8	2·04	2·04

It will be seen that the velocities are practically identical.

Series 2A.

Dealing next with the tests made on various arrangements of such bolsters, the nomenclature being similar to that adopted in the case of the cubes :

Case (*a*) single row,
,, (*b*) two rows,
,, (*c*) three rows,
,, (*d*) four rows,
,, (*e*)
,, (*f*)
,, (*g*)
,, (*h*)
,, (*i*) A pell-mell wall of bolsters, dropped at random, to form an embankment of crest-height approximately the same as in case (*g*).

The total depth of the embankment in cases (*e*), (*f*), and (*h*) was approximately 3·2 inches ; in cases (*g*) and (*i*) it was 4·5 inches. Table X shows the ratios of the velocities, $\bar{v}$, as measured with these bolsters, to the corresponding $\bar{v}$ with 2-inch cubes having $\rho' = 130{\cdot}4$ lb. per cubic foot :

TABLE X.

$\frac{h}{l}$	*a*	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*
1	0·94	0·73	0·82	0·76	0·91	0·78	1·38	0·71	* 1·03
2	0·87	0·78	0·80	0·73	0·93	0·76	1·17	0·70	1·02
5	0·85	0·81	0·74	0·69	0·96	0·73	0·94	0·71	0·99
10	0·80	0·74	0·73	0·73	0·92	0·67	0·82	0·69	0·98

* Initial movement.

There is a decided tendency for a mound constructed in regular fashion, as in case (*g*), to be more stable than a random formation, as in case (*i*), at small depths, but less stable at greater depths. The opposite is the case when cubical blocks are used, and pitot measurements, in fact, reveal that the velocity-distribution through the depth of water is different in the two examples of bolsters and cubes. Thus, reference to Table V (i) shows that with a pell-mell mound of cubes the surface velocity, v_s, is always rather lower than $\bar{v}$. On the other hand, with bolsters v_s is about 10 per cent. higher than $\bar{v}$ when $h/l = 1$ and 15 per cent. lower than $\bar{v}$ when $h/l = 10$.

Now the weight of a 3-foot concrete cube of density 150 lb. per cubic foot is 4,050 lb. A bolster having a density of 80·3 lb. per cubic foot and a length : diameter ratio of 3·54 : 2·1 would have dimensions of 5·66 feet by 3·37 feet (mean diameter) for a weight of 4,050 lb.

Table XI shows the mean velocities, $\bar{v}$, at which such a bolster is expected to move.

TABLE XI.

Total depth, D : feet.	6	9	12	18	33
$\bar{v}$: feet per second	6·2	6·4	6·8	7·4	8·7

The velocities shown in Table XI may be compared with those of Table IV; they are, of course, appreciably lower. On the other hand, a random mound of such bolsters compares much more favourably with one of cubes, as is demonstrated by Table XII.

TABLE XII.

Depth, h, over crest of mound : feet.	$\bar{v}$: feet per second.	
	3-foot cubes.	Bolsters.
3	4·9	4·6
6	6·1	5·6
15	9·6	8·1
30	12·6	10·9

Series 2A′. Tests on less flexible bolsters.

Bolsters were made of cotton bags containing stone chippings and sand, with a total weight per bolster of 1·25 lb. The chippings were similar to those used in the bolsters already described, but the total weight included 0·18 lb. of sand having a mean diameter of the order of 0·01 inch. These heavier bolsters were 5·2 inches long, with an average diameter of 2·1 inches, and their dry density was 119·8 per cubic foot. They were tested in water of between 10 and 11 inches total depth. Table XIII shows the velocities required to move them, relative to those for the lighter bolsters in the same depth of water:

TABLE XIII.

Arrangement.	$\bar{v}$ for heavy relative to $\bar{v}$ for light bolster.
Solitary	1·43
a	1·53
b	1·20
c	1·09
d	1·11
e	1·43
f	1·22
g	1·37
h	1·23
i (initial movement)	1·47

The solitary lighter bolster has been shown to have the same stability as a rectangular block of the same dry density (80·3 lb. per cubic foot) as itself and having a depth of 1·8 inch and a width of 2·45 inches. Suppose now that this block has a density $\rho' = 119{\cdot}8$ lb. per cubic foot (that is, the dry density of the heavier bolster): in 10·5 inches of water such a block would move at $\bar{v} = 3{\cdot}1$ feet per second, whereas the heavier bolster was found to roll over at 2·4 feet per second. On this basis, therefore, it would appear that the heavier bolster is relatively less stable than the lighter. This is due to the reduction in flexibility, which results in the heavier bolster retaining a much more cylindrical shape. Thus, the heavier bolster moves at almost exactly the velocity required for a block having $\rho' = 119{\cdot}8$ and a square section of 2·1 inches by 2·1 inches. Similarly, in the case of a mound constructed of such bolsters, one would expect the improved interlocking of the more flexible bolsters to enhance their stability.

In view of these considerations, it was decided to extend the investigation in order to determine the velocities necessary to disturb flat beds and mounds made of chippings alone.

Series 1B. Tests on aggregates of broken stone and of sand.

Five kinds of material were tried; their characteristics are summarized in Table XIV.

TABLE XIV.

Aggregate:	A.	B.	C.	D.	E.
Average weight per piece: lb.	0·104	0·0651	0·0216	0·00316	0·000356
Average of—					
Maximum length: inches	1·93	1·64	1·27	0·80	0·29
Maximum breadth: inches	1·45	1·25	0·91	0·49	0·22
Maximum thickness: inches	1·05	0·88	0·62	0·29	0·15
Ratios of—					
Length/thickness	1·84	1·86	2·05	2·76	1·93
Length/breadth	1·33	1·31	1·40	1·63	1·32
Breadth/thickness	1·38	1·42	1·47	1·69	1·46
$L + B/2l$	1·64	1·64	1·79	2·02	1·59
Length, l, of a cube of the same weight: inches	1·03	0·88	0·61	0·32	0·16
Specific gravity, σ'	2·64	2·64	2·64	2·64	2·56

Table XIV shows that grades A and B are geometrically very similar; grade C is not very dissimilar; whilst E lies between C and B. Grade D, however, has a much greater length/thickness ratio. Incidentally, a curious feature of the measurements is that in all cases the average thickness is almost exactly equal to the length of a cube of the same volume as the piece of aggregate. The effective density, in water, of grade E is 0·95 of that of the other grades.

All the materials had a high degree of uniformity of size; thus ten

successive groups of five pieces of grade A weighed 0·47 lb., 0·62 lb., 0·62 lb., 0·41 lb., 0·72 lb., 0·47 lb., 0·56 lb., 0·59 lb., 0·66 lb., and 0·50 lb., respectively or an average of 0·112 lb. per piece, in comparison with 0·104 lb. for 300 pieces. Two successive samples of grade E, each containing 120 pieces, weighed 19·2 grams and 19·4 grams, respectively.

In the tests on flat beds of such aggregates a layer of material was spread across the width of the flume ; the uniform thickness of the layer ranged between 1 inch and 2 inches in the different experiments, whilst the length of the bed, parallel to the axis of flow, ranged from 4 inches to 12 inches. At its ends the layer was tapered to the bottom of the flume at an angle of about 30 degrees. The observations led to the conclusion that the velocities, measured at half-depth, for movement of the aggregate were rather higher (10–15 per cent.) with a bed 12 inches in length than with one of 4 inches. On the other hand, the mean velocities $\bar{v}$ were sensibly the same in the two cases.

A difficulty experienced by all investigators in this field is to define when motion of the bed material does actually begin, or to describe the manner of motion once it has been initiated. In the present investigation, the following procedure was adopted : the velocity was increased gradually until one or two particles were seen to move. That velocity and the depth of water were noted. In general, it was observed that an appreciable increase in that velocity caused no further movement, but that presently several particles would move. After making the necessary observations at that stage, the flow was again increased until it was sufficient to produce general movement. This did not mean that at any instant the whole of the surface particles were in motion, but that, given time, it was evident that the whole surface would be scoured away, generally by bursts of activity at different points.

In analysing the data, an average has been taken of the first and second stages, designated the velocity at which the first few particles move. An average has similarly been worked out for the second and third stages, designated the velocity at which many particles move. As before, $\bar{v}$ denotes the mean velocity of flow, and h the depth of water over the bed of aggregate : l denotes the length of the side of a cube of volume equal to the average volume of an individual piece of material.

Figs. 5 (*a*) shows $\bar{v}$, at which the first few particles move, plotted against h/l. *Figs.* 5 (*b*) shows $\bar{v}$ for many pieces in motion, again plotted against h/l. From these graphs it is evident that there is a very pronounced effect of depth. Considering *Figs.* 5 (*a*) and 5 (*b*), the following data apply to a value of $h/l = 10$:

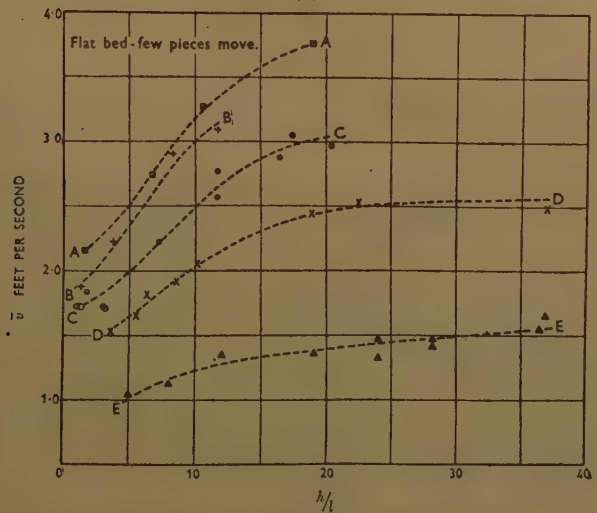
In Table XV, σ' denotes the specific gravity of the material, and σ that of water (assumed unity) : the results indicate that, for a given h/l ratio, $\bar{v}$ is very nearly proportional to $\sqrt{(\sigma'-\sigma)l}$, except in the case of grade D when the velocity required for movement is about 16 per cent. higher than might be inferred from the results with grade A. The reason for this

TABLE XV.

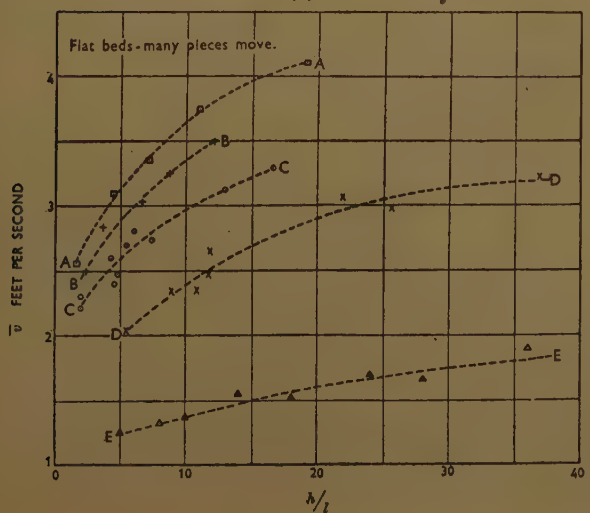
	B.	C.	D.	E.
$\bar{v}$ relative to $\bar{v}$ for grade A—				
first few move	0·87	0·78	0·64	0·39
many move	0·92	0·82	0·66	0·37
$\sqrt{(\sigma' - \sigma)l}$ relative to $\sqrt{(\sigma' - \sigma)l}$ for grade A .	0·92	0·77	0·56	0·39

Figs. 5.

(*a*)



(*b*)



appears to be largely provided by the fact, already noted, that the pieces of grade D are definitely dissimilar in shape to those of the other materials.

Arguing by analogy to the case of a rectangular block of length L and thickness l, the velocity for movement might be proportional to L/l, if L were measured parallel to the current. But in the case of a random bed of aggregate, some of the particles moved might be arranged with either the length L or the breadth B parallel to the direction of motion. Accordingly, a trial has been made of the effect of examining the results in terms of

$$\frac{\bar{v}}{\frac{L+B}{2l}\sqrt{(\sigma' - \sigma)l}}.$$

Again, it is evident that other factors might influence the movement of such aggregates, namely :—

The ratio of the hydraulic mean depth m to the simple depth h, since this might affect the velocity-distribution in the stream.

The uniformity modulus M of the material, which presumably will affect the interlocking of the particles. Here M is defined by plotting as ordinate the percentage, by weight, of particles finer than a certain grain-size against that grain-size as abscissa : dividing the resulting area which lies between the vertical axis and the curve into two parts, one above and the other below the 50-per cent. line, M becomes the ratio of the area below to that above the 50-per cent. dividing-line. For the Author's materials A–D, M is of the order of 0·90, whilst for material E it is 0·79.

In order to determine the effect of these variables, the results of two other investigations have been considered. Some years ago, experiments were made by Dr. T. L. Chou [1] in the Whitworth Engineering Laboratory at Manchester University. This work was carried out on level beds of material in a channel 6 inches wide.

The other investigation which has been considered is that made by the U.S. Waterways Experiment Station, Vicksburg, Mississippi, details of which were given in their Paper No. 17, dated January 1935 [2]. Experiments were made in a flume 27·8 inches wide, with bed slopes ranging between 0·0010 and 0·0045, and covered a variety of sands (specific gravity 2·65) ranging from 0·00809 inch to 0·160 inch in mean grain-size. The grain-size thus specified was based upon mechanical analysis " conducted in the

[1] " A Study of the Problem of Transportation of Bed Materials in River and Estuary Flow and of the Investigation of River Flow Problems by Means of Models." Thesis presented to the Victoria University of Manchester in application for the Degree of Ph.D., June 1936.

[2] " Studies of River Bed Materials and their Movement, with Special Reference to the Lower Mississippi River." (Mississippi River Commission Print. January 1935.)

RO-Tap testing sieve shaker"; accordingly it has been necessary to convert them to the dimension l defined in the present Paper as the length of a cube of the same weight. Now, experience in analysing a large number of different materials suggests that there is a connexion between the quantities $\left(\frac{L+B}{2l} \cdot \frac{B}{L}\right)$ and $\left(\frac{L+B}{2}\right)$, where L denotes the length and B the width of a piece of broken stone or of a grain of sand as seen in plan view. Thus, as $\left(\frac{L+B}{2}\right)$ tends to zero $\left(\frac{L+B}{2l} \cdot \frac{B}{L}\right)$ approaches unity; other values are very nearly as stated in Table XVI.

TABLE XVI

$\frac{L+B}{2}$: inches.	$\frac{L+B}{2l} \cdot \frac{B}{L}$
0·05	1·10
0·10	1·14
0·30	1·21
1·00	1·25

The Vicksburg materials are described as "sub-angular to sub-rounded", "angular to sub-rounded", and so on; and microphotographs illustrate the meaning of these definitions. It is possible, therefore, by measurements made on these microphotographs, to estimate the value of (L/B) for each of their materials, and hence, if it be assumed that L is approximately the same as the mean sieve grain-diameter, to assign a value to $\left(\frac{L+B}{2}\right)$. From this, the value of $\left(\frac{L+B}{2l} \cdot \frac{B}{L}\right)$ has been estimated, thus rendering l itself determinate.

On this basis, the properties of the materials are summarized below:

In Table XVII, the materials numbered 1 to 9 are those used in the Vicksburg tests; the rest are Dr. Chou's. It will be observed that the consideration of these other experiments increases the range of the study considerably. Moreover, in the Author's experiments, the ratio of h/m varied between about 1·03 and 4·29; in the appropriate Vicksburg results it attained a maximum of 1·23, whilst in Dr. Chou's work it ranged from 1·49 to 4·99.

In dealing with the Vicksburg observations, attention has been confined to those movements described as "weak", according to the following definition: "'Weak' movement indicates that a few or several of the smallest sand particles are in motion, in isolated spots, and in countable numbers. By countable is meant that by confining the field of observation to, say 1 square centimetre, the particles in motion can be counted by the observer." This kind of motion might, therefore, be expected to lie between what has been described in the present Paper as "first few particles" and

TABLE XVII.

Material.	$\frac{L+B}{2l}$	l: inches.	M	σ'
1	1·51	0·0129	0·280	2·65
2	1·51	0·0119	0·439	"
3	1·44	0·0124	0·539	"
4	1·54	0·0109	0·406	"
5	1·54	0·0104	0·438	"
6	1·49	0·00776	0·643	"
7	1·47	0·00707	0·525	"
8	1·49	0·00456	0·560	"
9	1·66	0·0819	0·566	"
α	1·38	0·00384	0·745	2·52
β	1·47	0·0275	0·834	2·64
γ	1·44	0·00736	0·688	2·64 (5)
σ	1·46	0·0139	0·703	2·64 (3)
ϵ	1·62	0·0760	0·790	2·63 (3)
Powdered pumice.	1·61	0·00775	0·755	2·016
Powdered emery.	1·36	0·00607	0·812	3·89

" many particles " move. The types of movement described by Dr. Chou are " initial ", " occasional ", " some ", and " general but without ripple-formation."

Figs. 6, Plate 1, show the results plotted in the form of

$$y=\left(\frac{\bar{v}\sqrt{h/m}}{\frac{L+B}{2l}\sqrt{(\sigma'-\sigma)\frac{l}{M}}}\right)\text{ against } x=\frac{1}{\left(\frac{L+B}{2l}\right)^2}\cdot\frac{1}{\sigma'-\sigma}\cdot\frac{h}{l}\cdot\frac{h}{m}\cdot M,$$

for which purpose the effective grain-size is treated as $\left(\frac{L+B}{2l}\right)^2(\sigma'-\sigma)\frac{l}{M}$. The Author's results for both cases, " first few " and " many particles " move, are included.

In preparing this diagram, a difficulty arose concerning the bed-slopes of the Vicksburg tests; but it was found that if the actual $\left(\frac{h}{l}\cdot\frac{h}{m}\cdot M\right)$ were increased by an amount equal to $7{\cdot}5y^4S^{\frac{1}{2}}$, where S denotes the bed-slope, then the results obtained with all their bed materials and slopes became consistent, not only among themselves, but also in relation to the level-bed experiments of Dr. Chou and the Author. Accordingly, that adjustment has been made in Figs. 6, Plate 1, from which it will be seen that the plotted points lie almost entirely between two curves corresponding to the two types of motion " first few particles " and " many particles " move.

The equations of these curves are respectively

$$y = 1{\cdot}10x^{0{\cdot}28} \quad \ldots \quad (14)$$

and

$$y = 1{\cdot}59x^{0{\cdot}28} \quad \ldots \quad (15)$$

or $$\bar{v} = K\left(\frac{L+B}{2l}\right)^{0 \cdot 44}(\sigma' - \sigma)^{0 \cdot 22} l^{0 \cdot 22} M^{-0 \cdot 22} h^{0 \cdot 06} m^{0 \cdot 22} \quad . \quad . \quad (16)$$

where $K = 1{\cdot}10$ for movement of the first few particles
and $K = 1{\cdot}59$ for movement of many particles.

Here, $\bar{v}$ is in feet per second, and all length dimensions are in inches.

In comparatively broad channels, $h = m$, and then

$$\bar{v} = K\left(\frac{L+B}{2l}\right)^{0 \cdot 44}(\sigma' - \sigma)^{0 \cdot 22} l^{0 \cdot 22} M^{-0 \cdot 22} h^{0 \cdot 28} \quad . \quad . \quad (17)$$

It is of interest to compare the velocities required to disturb a flat bed of 3-foot cubes ($\sigma' = 2{\cdot}40$, say) weighing 4,050 lb., with irregular pieces of the same l and weight. The values for the cubes, laid in regular fashion, are based upon Table V (h), whilst those for the aggregate are taken from equation (17), assuming $K = 1{\cdot}10$, $\dfrac{L+B}{2l} = 1{\cdot}50$, and $M = 0{\cdot}90$.

TABLE XVIII.

Depth over bed, h: feet.	$\bar{v}$ for initial movement: feet per second.	
	3-foot cubes.	Aggregate.
3	10·8	8·8
6	12·2	10·6
15	14·9	13·7
30	16·9	16·6

Except at low depths, therefore, there is little to be gained by using the regularly-shaped pieces laid in regular rows and layers.

Series 2B. Tests on Mounds of Aggregates A, B, C, D.

With aggregate A, the mound was made 4 inches high and 9 inches wide at the base; it stretched across the full width of the channel. The mounds of smaller aggregates were of similar, that is, correspondingly smaller, cross-section, and it was impracticable to try material E because of the small dimension of mound which would have been needed to be in geometrical similarity with the others.

Observations were taken just after the movement of one or of a very few pieces from the crest of the mound: usually the first piece to move was one situated near the centre of the flume and having its longest dimension normal to the stream. Following this, the flow was increased and, in general, particles were seen to start moving up the side facing the current. Further observations were taken when the crest had been so eroded as to be reduced to between two-thirds and three-quarters of its initial height.

Analysis shows that the effect of the ratio of the actual depth to the

hydraulic mean depth is not so marked as in the case of the flat beds, and no doubt this is due to the more evenly distributed velocity across the width of the stream as it flows over the mound.

Let
$$y_1 = \frac{\bar{v}(h/m)^{\frac{1}{4}}}{\sqrt{(\sigma' - \sigma)\dfrac{l}{M}}} .$$

let
$$x_1 = \frac{1}{\sigma' - \sigma} \cdot \frac{h}{l} \cdot \frac{h}{m} \cdot M ;$$

let
$$y_2 = \frac{\bar{v}(h/m)^{\frac{1}{4}}}{\dfrac{L+B}{2l}\sqrt{(\sigma' - \sigma)\dfrac{l}{M}}} ;$$

and let
$$x_2 = \frac{1}{\left(\dfrac{L+B}{2l}\right)^2} \cdot \frac{1}{\sigma' - \sigma} \cdot \frac{h}{l} \cdot \frac{h}{m} \cdot M ;$$

then *Figs.* 7 (*a*) shows y_1 plotted against x_1 for the movement of the first one or two pieces, whilst *Figs.* 7 (*b*) shows y_2 against x_2 for the flattening of the crest. That no $\left(\dfrac{L+B}{2l}\right)$ term appears to be necessary in y_1 is presumably due to the fact that whatever the shape of the particles they are comparatively delicately poised on the crest of a mound. The effect of shape is not, however, completely absent, since l itself depends upon the shape.

According to *Figs.* 7 (*a*) *and* 7 (*b*),

$$y_1 = 1{\cdot}30\ x_1^{0{\cdot}23} \text{ for movement of the first few pieces} \quad . \quad (18)$$

and
$$y_2 = 1{\cdot}37\ x_2^{0{\cdot}23} \text{ for flattening of the crest} \quad . \quad . \quad . \quad . \quad . \quad (19)$$

or, for initial movement,

$$\bar{v} = 1{\cdot}30\ (\sigma' - \sigma)^{0{\cdot}27} l^{0{\cdot}27} M^{-0{\cdot}27} h^{0{\cdot}21} m^{0{\cdot}02} \quad . \quad . \quad . \quad . \quad . \quad (20)$$

whilst for flattening of the crest,

$$\bar{v} = 1{\cdot}37 \left(\frac{L+B}{2l}\right)^{0{\cdot}54} (\sigma' - \sigma)^{0{\cdot}27} l^{0{\cdot}27} M^{-0{\cdot}27} h^{0{\cdot}21} m^{0{\cdot}02} \quad (21)$$

$\bar{v}$ is measured in feet per second and the length dimensions are in inches.

In comparatively broad channels, $\bar{v}$ is proportional to $h^{0{\cdot}23}$.

Suppose that a comparison is to be made with the example of a mound of 3-foot cubes, each weighing 4,050 lb. ($\sigma' = 2{\cdot}40$). If aggregate of the same weight be used (that is, $l = 36$ inches; $\sigma' = 2.40$), and if this aggregate has a similar shape to that of material D, then Table XIX provides a direct comparison of the stability of the mounds.

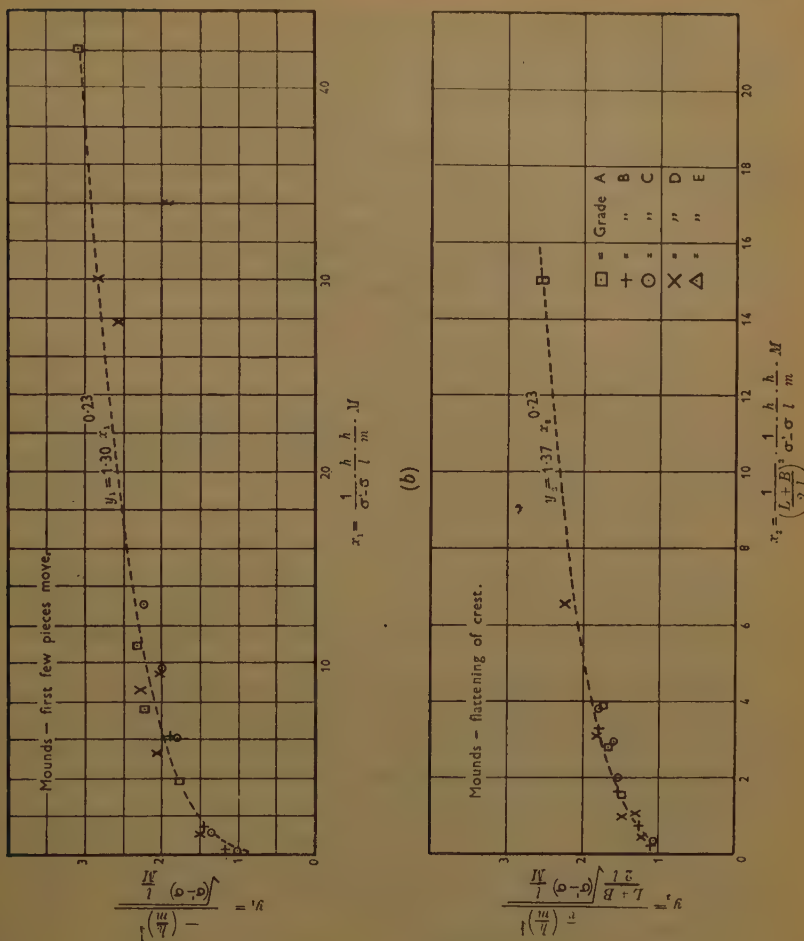


TABLE XIX.

Depth over crest, h: feet.	$\bar{v}$ for initial movement: feet per second.	
	3-foot cubes.	Aggregate, $l = 3$ feet, $M = 0{\cdot}90$.
3	5·1	8·8
6	6·3	10·3
15	9·4	12·7
30	12·5	14·9

At low depths, therefore, a great deal can be gained by using pieces of irregular shape.

It is also important to make a comparison with the results previously obtained for a mound of bolsters. Table XII was prepared for bolsters 5·66 feet long and 3·37 feet in diameter, weighing 4,050 lb. The more flexible bolsters used in the laboratory tests measured 3·54 inches by 2·1 inches, or $\frac{1}{19 \cdot 2}$ of the dimensions of the full-size bolsters visualized in Table XII. The laboratory bolsters were filled with aggregate D, for which $l = 0{\cdot}32$ inch, so that the corresponding size of aggregate for the 4,050-lb. bolster would be $l = 0{\cdot}32$ by 19·2, or 6·15 inches. Again, aggregate D has $\sigma' = 2{\cdot}64$, $M = 0{\cdot}90$. Each piece for which $l = 6{\cdot}15$ inches would weigh 22·1 lb. Table XX shows the comparative data.

TABLE XX.

Depth over crest, h: feet.	$\bar{v}$: feet per second, for initial movement of mound of:		
	3-foot cubes (4,050 lb.).	Bolsters (4,050 lb.).	Aggregate, $l = 6{\cdot}15$ inches (22·1 lb.).
3	5·1	4·6	5·7
6	6·3	5·6	6·7
15	9·4	8·1	8·2
30	12·5	10·9	9·7

For relatively small depths of water, the stone chippings are preferable to either the much heavier cubes or the bolsters; moreover, their comparative stability at great depths is considerable. But of course their success in practice would depend upon the possibility of building up such a mound in periods of quiescent currents.

MOVEMENT OF SAND IN TIDAL ESTUARIES AND IN THEIR MODELS.

The question is often raised as to how it is possible for sand to be transported in a tidal model when the size of the sand is of the same order as that in the actual estuary whilst the velocities, being proportional to the square root of the vertical scale, are so much lower. The present investigation may throw additional light on this subject. To take an actual example, consider the model of the Severn estuary having a vertical scale of 1 : 200[1]. The mean diameter of the sand in the estuary is about 0·0089 inch, whilst that finally adopted in the model was 0·0070 inch. These dimensions were based upon microscope measurements and correspond approximately to $\frac{L+B}{2}$. For the estuary sand, $\sigma' = 2{\cdot}60$,

[1] A. H. Gibson: "Construction and Operation of a Tidal Model of the Severn Estuary", H.M. Stationery Office, 63–78–2; 1933.

$\frac{L+B}{2l} = 1{\cdot}27$, and $M = 0{\cdot}60$ approximately, whilst the corresponding values for the model sand are $\sigma' = 2{\cdot}62$, $\frac{L+B}{2l} = 1{\cdot}25$, and $M = 0{\cdot}75$.

According to equation (16), the velocities (feet per second) required for the first few grains to move on an initially level bed in a straight channel of rectangular cross-section would be

$$\bar{v} = 0{\cdot}51 h^{0{\cdot}06} m^{0{\cdot}22} \text{ for the estuary sand} \quad . \quad . \quad . \quad . \quad (22)$$
$$\bar{v} = 0{\cdot}46 h^{0{\cdot}06} m^{0{\cdot}22} \text{ for the model sand} \quad . \quad . \quad . \quad . \quad (23)$$

h and m being measured in inches.

Considering these expressions in relation to tide and current data, it is found that on a spring tide, the velocities off Avonmouth, in the actual estuary, are sufficient for movement of the sand-bed during about 75 per cent. of the flood and 75 per cent. of the ebb. In the model, they are sufficient during about 20 per cent. of the flood and 30 per cent. of the ebb; but such movement would be enough to create small ripples and so to reduce the velocities required for further movement at least to the order of those associated with a mound. In fact, using the formula for initial disturbance of a mound, it appears that, in the model, velocities would be sufficiently high during about 40 per cent. of the flood and 40 per cent. of the ebb.

A similar treatment of the portion of the estuary near Sharpness, on the basis of the level-bed formula, indicates that in the estuary itself, the velocities are sufficient during about 75 per cent. of the flood and 75 per cent. of the ebb, whilst in the model the corresponding percentages are 45 and 30. But if the " mound formula " be applied, the percentage-times in the model are at once increased to 55 on the flood and 40 on the ebb: in practice, these times might be more nearly equalized by the appreciable bed-gradient in this part of the model coupled with the vertical exaggeration of the scale.

All these conclusions are based upon the formulas derived from bed movement in straight, parallel-sided channels. It is clear that in an estuary, with its irregularities of coastline and cross-sectional area, the velocities required for movement of a sand-bed are likely to be reduced considerably, having regard to the pronounced eddy-formation created by sudden changes of section.

The tabulated results of the straight flume tests of the Vicksburg laboratory for a sand of (sieve) grain-size 0·00809 inch reveal the following interesting facts:

With a depth of 0·60 inch, a velocity of 0·55 foot per second caused weak motion on a " smooth " bed, as also did 0·75 foot per second when the depth was 0·82 inch: but with the bed in a condition of " general riffles ", and with a much greater depth (2·60 inches), a velocity of only 0·50 foot

per second caused general movement. Had the bed been still in a flat or smooth state, a velocity of 0·70 foot per second would have given only initial motion at this depth of 2·60 inches.

To take another example, with a different bed-slope, weak movement of the "smooth" bed was observed with $h = 0{\cdot}41$ inch and $\bar{v} = 0{\cdot}64$ foot per second, but with the bed in a state of general ripple-formation, $\bar{v} = 0{\cdot}45$ foot per second was accompanied by general motion when h was equal to 1·15 inches. Here again, a velocity of 0·55 foot per second would have produced only initial motion if the bed had remained smooth. In practice, this velocity (0·55) proved, owing to ripple-formation, to give general motion with h as high as 1·5 inch.

The mechanism of the action in the model appears then to be this: provided that at some stage of the tidal cycle, the currents are strong enough to move the material at all, the bed will (if of the kind usually adopted in such experiments) take the form of local ripples or sand-ridges. Once these have been created, the movement will immediately tend to become more intense and the period during which the bed will persist in active movement will be very appreciably prolonged.

A further point of interest may be mentioned: in the tests on the Severn model, it was discovered [1] that for materials of a given L/B ratio, the lateral and longitudinal gradients assumed by the bed were proportional to $(d^{-0{\cdot}287}\rho_1{}^{-0{\cdot}262})$, where $d = \dfrac{L+B}{2}$, and ρ_1 denotes the effective weight in water of unit volume of the material. It is remarkable that the numerical values of these indexes are almost exactly the same as those belonging to l and $(\sigma' - \sigma)$ in equations (20) and (21). This fact supports the idea that $(\sigma' - \sigma)^{0{\cdot}27} l^{0{\cdot}27} M^{-0{\cdot}27}$ may be a criterion of the comparative behaviour of different materials in a tidal model. Let this criterion be designated C: in Table XXI values of C are given for a number of the materials tried in the Severn investigation:

TABLE XXI.

Material.	$\dfrac{L+B}{2}$: inches.	l: inches.	σ'	C
120-mesh silica sand	0·00582	0·00485	2·63	0·291
80- " " "	0·00700	0·00560	2·62	0·304
50–80 Cobham sand	0·0100	0·00788	2·64	0·330
120-mesh emery	0·00505	0·00336	3·89	0·303
80- " "	0·00825	0·00606	3·89	0·354
Pumice A	0·0122	0·00754	1·99	0·289
" B	0·0153	0·00957	1·95	0·305
" C	0·0192	0·0123	1·90	0·311
Severn estuary sand	0·00893	0·00702	2·60	0·340

[1] A. H. Gibson, *loc. cit.*; Appendix D.

The material which gave the best overall reproduction was the 80-mesh silica sand ($C = 0{\cdot}304$): but the 120-mesh emery ($C = 0{\cdot}303$) and pumice B ($C = 0{\cdot}305$) behaved in very similar fashion. The 120-mesh silica sand ($C = 0{\cdot}291$) was also good, but exhibited a tendency to be carried too easily upstream, whilst pumice A ($C = 0{\cdot}289$) suffered also from this defect. The conclusion is therefore drawn that C furnishes a remarkably sensitive criterion of behaviour of materials having very different sizes and densities.

SUMMARY.

A close estimate of the mean current-velocity required to overturn a rectangular prism may be made with the aid of equation (13); at the instant of overturning, the greatest velocity between the top of a cubical block and the water-surface is $\sqrt{2g \cdot \frac{\rho' - \rho}{\rho} \cdot l}$ feet per second, using foot, pound, and second units for g, ρ', ρ, and l.

The stability of a wall or mound of cubical blocks is discussed under heading *Series 2*, whilst *Series 1A, 2A, and 2A′* relate to experiments on more or less flexible bolsters. It is shown that whilst an isolated bolster is less stable than an isolated cubical block of the same weight, yet the difference in the case of a random mound is far less marked. Emphasis is laid upon the effect of the degree of flexibility of such bolsters and upon their property of interlocking.

Tests on flat beds and on mounds of various aggregates are described under headings *Series 1B* and *2B*; it is shown that higher velocities are resisted by embankments of irregularly-shaped pieces of stone than by cubical pieces of the same weight, and that in comparatively shallow water a mound of chippings is more stable, once it has been constructed, than if the chippings were enclosed in cages to form bolsters of very considerable weight.

Experiments on beds of various materials, carried out by the U.S. Waterways Experiment Station at Vicksburg, Mississippi, and by Dr. T. L. Chou at Manchester University, are discussed in some detail in relation to the present investigation, leading to conclusions based upon experiments with materials ranging from an equivalent cube-length of 0·00384 inch to 1·03 inch and having specific gravities ranging from 2·016 to 3·89.

The Paper concludes with a note on the apparent mechanism of the bed movement in a tidal model.

ACKNOWLEDGEMENT.

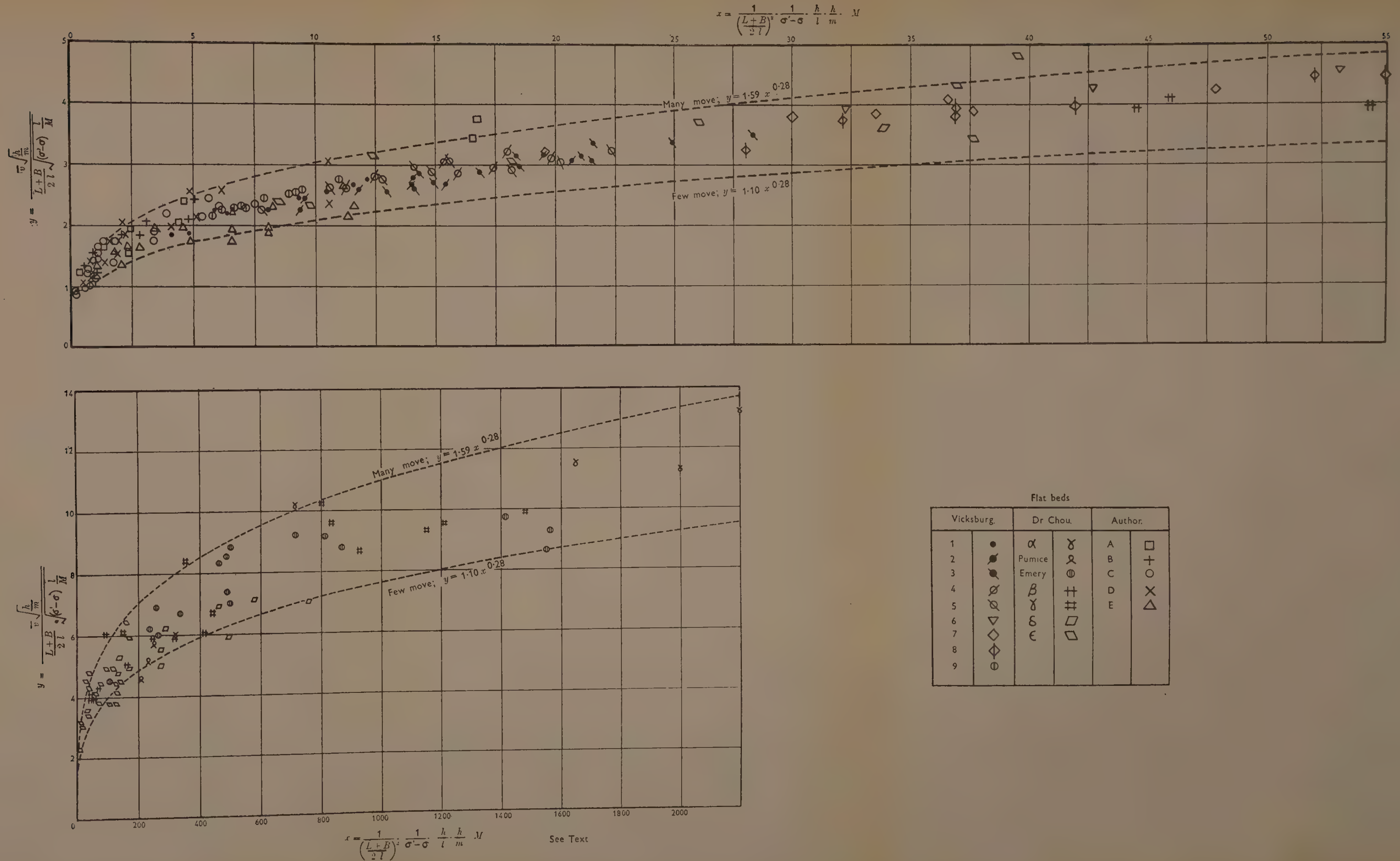
The Author wishes to record his sense of indebtedness to Professor

A. H. Gibson, D.Sc., LL.D., M. Inst. C.E., for the facilities placed at his disposal in the Whitworth Engineering Laboratories at Manchester University, and for much helpful criticism.

The Paper is accompanied by seven sheets of drawings, from which Plate 1 and the Figures in the text have been prepared.

AN INVESTIGATION OF THE STABILITY OF BED MATERIALS IN A STREAM OF WATER.

FIGS. 6.



ORDINARY MEETING.

10 February, 1942.

Professor CHARLES EDWARD INGLIS, O.B.E., M.A., LL.D., F.R.S., President, in the Chair.

On the motion of the President, it was resolved :—

That the President and Council and the Members of The Institution of Civil Engineers deeply regret the death of His Royal Highness Arthur, Duke of Connaught, K.G., P.C., G.B.E., who had been an Honorary Member of The Institution since May, 1872.

The President put to the meeting a recommendation by the Council that the Right Reverend Paul Fulcrand Delacour de Labilliere, Dean of Westminster, be elected an Honorary Member of The Institution.

The recommendation was agreed to by acclamation.

The Council reported that they had recently transferred to the class of

Members.

Arthur Ethelbert Griffin, M.C.
Stuart Stanley Harrison.
John McGregor Morris.
Hubert Pryce-Jones, M.Eng. (*Liverpool*).
John Edmund Sandham, B.Sc. (Eng.) (*Lond.*).
Francis White, B.Sc. (Eng.) (*Lond.*).

And had admitted as

Students.

Paul Hugh Finden Andrew.
William Henry Arch.
Cecil Thomas Bamford.
Michael Harington Barnard.
Frank Baron, B. Eng. (*Liverpool*).
Henry Leslie Louis John Barter.
Alan Bernard Bellenger.
Ernest Walter Bennett.
John Black.
David Roy Boulter.
Richard Owen Boxer.
Colin Patrick Bradley.
James Oliver Levi Brass.
Douglas George Bromfield.
John Brownlie.
John Alan Buckby.
Alan William Bullett.
Philip Darnley Burns.
John Alan Butcher.
Alan Chapman.
Colin Frederick Clarke.
Harold Lawton Cliffe.
Charles Eric Copeland.
Frank Crowther.
Tom Dupe.
Albert Wortley Emmett.
John Richard Fairbank.
John Harloe Fleming.
Robert Philip Fletcher.
Donald James Fuggle.
John Herbert Gaine.
Frank Montagu Lucas Glaysher.
Shirley Graham Greenway Goode
Alexander Ramsay Grant.
Norman Sydney John Grassam.
Frederick Grover.

Douglas Ian Halstead.
Kenneth John Handscombe.
Robert Hastie.
Stephen Michael Hawes.
Peter William Helson.
Geoffrey Rowland Hill.
Douglas William Holder.
Julius Kenneth Beville Hutchinson.
Herbert Wyndham Jenkins.
Donald Frazer Johnson.
Edmund Wilfred Jones.
John Clark Judson.
Henry Joseph Ivan Kraly.
Murdoch Lamont.
William Robert Laurenson.
David Courtney Leonard.
Sue-Leong Low, B.A. (*Cantab.*).
Thomas Lynch.
John McClure.
Rowland Johnson Mainstone.
Peter William Malin.
Louis Robert Martin.
Cedric Mellor.
Eric Mellor.
John Terry Milton.
George Ian Munro.
John Robert Hales Oddy.
Denis Basil O'Neill.
Harold Ordman.
Sydney Bernard Ovis.
Brian Kitsell Parnell.
William Tennant Patterson.
Norman William Phillips.
Harry Frank Pitcher.
Hugh James Platts.
Frederick Forrest Poskitt.
Llewelyn Charles Livingston Pryce.
John Round.
Alexander Christer Edward Sandberg.
Patrick George Sands.
Victor Lionel Frederick Sapsworth.
Ian Harry Saunders.
John Llewelyn Arthur Selby.
Derek Smeaton.
David William Smith.
Norman Crawford Smyth.
Richard Smyth.
Alec Thomas Sneller.
William Gordon Walter Spink.
Cedric Charles Stainer.
John Leonard Steven.
John Darling Stevenson, B.A. (*Cantab.*).
Ronald Wilson Stirling.
Ian Woodhouse Taylor, B.Sc. (Eng.) (*Lond.*).
Neville Owner Taylor.
Leonard Herbert Tenneson, B.Sc. (*Eng.*) (*Lond.*).
David Morris Thomas.
Richard Eric Tuckwell.
James William Richards Twyman.
Keith Miles Vine-Lott.
Albert James Ward.
Edward Gordon Warwick-Oliver.
William Arthur James Webb.
Lawrence West.
Robert Theo West.
William Arthur Wilkins.
Ralph Neville Williams.
John Anthony Wilshere.
Andrew Tufts Wilson.
Roy Vernon Wilson.
Harry Leslie Yeadon.

The Scrutineers reported that the following had been duly elected as

Members.

Major-General Alan George Bingham Buchanan.
Frederick Harold Dunn Page.

Associate Members.

Edward William Martin Britain, B.Sc. (Eng.) (*Lond.*).
Douglas Corbett Haddow, Stud. Inst. C.E.
Douglas Leatherland, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Clifford Raymond Lee, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Archibald Parker Macdonald, B.Sc. (*Dalhousie*), Stud. Inst. C.E.
Angus Sim Mackay, B.Sc. (*Aberdeen*), Stud. Inst. C.E.
William Rutherford, B.Sc. (*Durham*), Stud. Inst. C.E.
William Forrest Thacker, Stud. Inst. C.E.
James Maurice Whitaker, Stud. Inst. C.E.
John Whitaker Whitaker, Stud. Inst. C.E.
Jack Widdowson, Stud. Inst. C.E.

The following Papers were submitted for discussion, and, on the motion of the President, the thanks of the Institution were accorded to the Authors.

Paper No. 5292.

"Soil Mechanics and Site Exploration." * †

By Leonard Frank Cooling, M.Sc.

TABLE OF CONTENTS.

Introduction.

To the civil engineer there are few problems more likely to give rise to uncertainty than those relating to the design and construction of earthworks and foundations. Such problems involve a consideration of those uncemented geological deposits located above the rock, such as gravels, sands, silts, and clays, which to the engineer are all included in the term "soil." Constructional operations involve changes in the stress conditions in the soil. The soil reacts to the stress changes, and the engineer is concerned with how these reactions are likely to influence his structure and whether they will give rise to difficulties during construction.

"Soil mechanics" is the name which has been given to the study of the reaction of soils to changing stress conditions, and during the past 16 years considerable advances have been made in the understanding of earthwork and foundation problems. The development of this branch of engineering has, for the most part, been due to Professor Karl von Terzaghi, M. Inst. C.E., and his associates, and in his "James Forrest" Lecture in May 1939 [1] he gave a stimulating survey of its achievements and possibilities. He pointed out that the chief function of soil studies is to supplement and guide the practical experience and judgment of the engineer by giving a theoretical insight into what happens in the field. Soils and site conditions are too variable to allow of the formulation of hard and fast laws, but the methods of soil mechanics are capable of giving substantial help to the engineer by at least indicating what is likely to happen in the practical case and often by allowing quantitative estimates to be made of the probable movements and pressures in particular problems.

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† Correspondence on this Paper can be accepted until the 15 July 1942, and will be published in the Institution Journal for October, 1942.—Sec. Inst. C.E.

[1] "Soil Mechanics—A New Chapter in Engineering Science", Journal Inst. C.E., vol. 12 (1938–39), p. 106. June 1939.

The exploration of the foundation soils at a particular site is often a necessary preliminary to constructional work, since only by this means is it possible to obtain the information necessary for the purposes of foundation design. It is also essential where questions of remedial measures are concerned. No two sites are exactly the same; differences are met with in the nature, thickness, and sequence of soil strata and in the position and possible variation of the ground-water-level. Even when a site shows a deep and apparently homogeneous foundation stratum, detailed exploration may be necessary, since even in such strata it is found that the soil properties may vary to an important extent at least in a vertical direction and often in a horizontal direction as well. The amount of attention required in exploring a site varies, of course, with different problems and depends upon such factors as the extent of the site and its probable difficulties, the size and type of structure, and whether it is important or presents special features. In addition, a good deal depends upon the experience and judgement of the engineer. If the soil conditions at the site and the type of structure are not very dissimilar from those with which he is familiar, little or no difficulty may be presented. The visual examination of samples from test-pits or borings, or in special cases a few loading tests, may then suffice to give the information required.

However, it is in those cases where the engineer meets soil conditions which are radically different or where the structure presents novel features that the methods of soil mechanics can be of greatest value.

The purpose of the present Paper is to outline the theoretical and practical considerations which soil mechanics indicates should be taken into account in the examination of sites for engineering structures. It therefore deals more with the sort of information which is required from soil surveys than with a detailed description of methods for carrying out site exploration. The use of the methods of soil mechanics is illustrated with reference to some of the practical problems which have been studied at the Building Research Station.

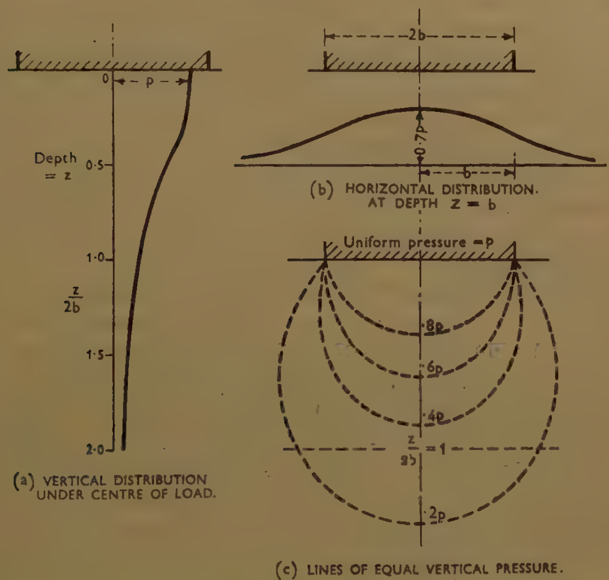
Depth and Extent of Exploration.

Broadly speaking, a soil survey should give information concerning the type, consistency, thickness, and sequence of soil strata and should indicate the degree of variation in both the vertical and the horizontal directions. It should include all those strata the deformation of which is likely to influence the behaviour of the structure. The depth and extent of the exploration necessary in such a survey can be estimated from theoretical and practical considerations; the following is a brief summary of the factors concerned.

Depth of Exploration: Theoretical Considerations. Changes in stress-distribution are usually brought about by structural loads applied at or near the surface, or by the removal of load by excavation. However, it must also be borne in mind that changes in the ground-water-level, arising

from the constructional operations or from adverse weather conditions, can modify the stress-distribution appreciably by introducing hydrostatic uplift and seepage pressures, and neglect of these factors may lead to serious trouble. The stress-distribution in the soil arising from structural loads is calculated by elastic theory on the assumption that the soil behaves as a homogeneous elastic medium. Whilst this assumption is not strictly true, it is considered to be sufficiently accurate for the purpose. In foundation problems the distribution of the normal stress on a horizontal

Figs. 1.



Distribution of Vertical Pressure: Uniformly Loaded Square Area.

plane and the distribution of the maximum shear stress most frequently require consideration.

The distribution of the vertical normal stress is needed in settlement problems, and its calculation gives rise to results which can be expressed by the well-known "bulb of pressure", the most important features of which are illustrated in *Figs. 1.* From the point of view of soil exploration it is important to note that under the loaded area the stress is still about one-fifth of the applied pressure at a depth equal to about one and a half times the breadth of the loaded area. Hence the larger the loaded area the deeper is its influence felt. Moreover, if a number of loaded footings are in close proximity, the effects from each are additive and the separate pressure-bulbs merge into one large pressure-bulb. Consequently if spread footings are closely spaced the stress may still be about one-fifth of the applied pressure at a depth equal to one and a half times the breadth

of the building. The calculation of these stresses in particular problems is facilitated considerably by the use of tabular coefficients such as those derived by N. M. Newmark [1].

The consideration of the distribution of maximum shear stress is important in stability problems, where there is danger of actual rupture of the soil. Such cases are usually associated with conditions of unsymmetrical loading on the soil, as in dams, quay walls, retaining walls, embankments, cuttings, etc., but excessive loads, even when the loading is symmetrical, may result in large shear deformation. The distribution of maximum shear stress has been derived by Carothers [2, 3], Jürgenson [4], and others. *Figs. 2* show the results for two important practical cases, namely, the long strip footing with (*a*) uniform loading and (*b*) "triangular" loading. These results indicate that the maximum shear stresses are appreciable at depths of one-half to one and a half times the breadth of the footing. Theoretical considerations therefore suggest that in most problems it is necessary to explore the foundation soils to much greater depths than are usually contemplated, and in some cases where compressible soils are present it may be necessary to explore to at least one and a half times the breadth of the structure.

Practical Considerations. The actual depth to which exploration should be carried may be influenced to an important extent as the result of information either available beforehand or obtained during preliminary investigation. For instance, geological evidence such as the published work of the Geological Survey, or records of borings in the neighbourhood, may give a valuable lead as to the soil strata likely to be encountered at a particular site and their variability as regards thickness and soil type. In special cases such information may point to uniform and favourable conditions at a depth, and then it may be possible to dispense with deep exploration and to limit the study to the soil within the more highly stressed zones. In general, however, the soil strata should be explored to the depths indicated by theory, although early results may suggest necessary modifications. The type, consistency, thickness, and sequence of the soil strata should be noted and the important strata recognized. A few of the major points requiring attention may be briefly indicated as follows. It is obvious that soft compressible strata, such as soft clays, peat, or soft silt, represent a potential source of trouble: they have a low shear strength and are likely to give large settlement movements by consolidation under the influence of the vertical normal stress. Soft clays may be particularly

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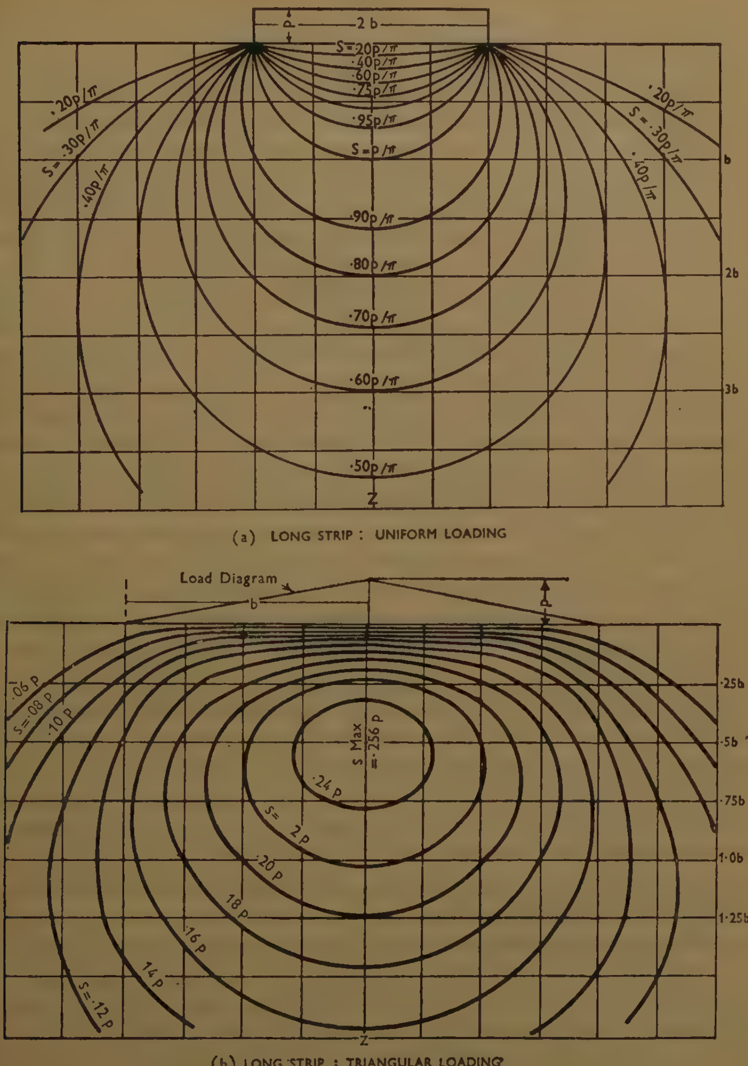
[1] N. M. Newmark, Univ. Illinois Eng. Expt. Station Circular No. 24, 1935.

[2] S. D. Carothers, "Plane Strain—the direct determination of stress", Proc. Roy. Soc., Series A, vol. xcvii, p. 110. 1920.

[3] S. D. Carothers, "The Elastic Equivalent of Statically Equipollent Loads", Proc. of Internat. Math. Congress, Toronto, vol. ii, p. 519. 1934.

[4] L. Jürgenson, "The Application of Theories of Elasticity and Plasticity to Foundation Problems." Journal Boston Soc. Civil Engineers, vol. 21, p. 206. July 1934.

Figs. 2.



Distribution of Maximum Shearing Stress.

troublesome owing to the fact that their shear strength tends to increase very slowly under applied loads, and in most cases no increase in shear strength during the construction period can be relied upon. The position of a soft layer relative to the surface is important, and the more highly stressed it is the more likely is it to cause trouble. Frequently a deep-seated soft layer has been the cause of undesirable movements owing to the

fact that, its presence being unsuspected, the permissible loading, judged on the basis of the more favourable strata located above it, has been high enough to impose appreciable stresses in the soft layer. The thicker the soft layer the greater will be the ultimate movement due to consolidation, although it may take a long time for the maximum movement to be attained. The sequence of soil strata may also be very important, especially as regards the position of permeable layers. If a layer of soft clay is sandwiched between two layers of permeable sand, the consolidation movements may take place many times more quickly than when the adjacent strata are relatively impermeable. In this connexion it is necessary to establish whether the sand is part of an isolated pocket or is in a continuous sand stratum. A loosely packed sand or gravel stratum may be a source of trouble, especially if it is likely to be subjected to vibrations or high seepage pressures. Vibration tends to make a loose sand assume a denser packing, and with a saturated sand part of the stress is thereby transferred temporarily from the grains to the water, thus reducing its shear strength. A rapid increase in seepage forces may also produce a similar effect. Finally, information should be obtained concerning the position of the ground-water-level and the hydrostatic pressure in the various soil layers. (Emptying the water from the borehole and observing the rate of change of the water-level may give valuable information on this point.) Artesian pressure in a sand layer below foundation-level may have an important influence on structures erected above it, or may give rise to serious trouble where deep excavations are made.

Extent of Exploration. The soil survey should be sufficiently extensive to furnish an idea of the degree of variation of the soil in a horizontal direction. This is particularly important in problems relating to building foundations, where the differential movement of one part of the structure relative to another is an important consideration. Such problems require that the exploration should indicate the variation in thickness of the various soil layers and permit assessment of the degree of variation in the soil properties under different parts of the structure. It is impracticable to give definite indications as to the amount of work required, which will vary according to the site and problem, and information obtained during the early part of the survey may suggest the need for more detailed exploration or otherwise. However, the survey should at least reveal the general variation in thickness and properties of the important soil layers in two horizontal directions over the area occupied by the structure.

Methods of Exploration.

A number of different methods of soil exploration are available, and they can be divided very broadly into two groups, namely (*a*) those which give positive evidence of the soil conditions; (*b*) those which give indirect evidence from which, in certain cases, the soil conditions can be inferred.

Direct Methods. In view of the many factors which enter into foundation problems, it is obviously desirable in the general case to rely on direct methods to furnish the information necessary for design purposes. This group of methods includes test-pits and borings, both of which permit recording of the sequence of soil strata and observation of the ground-water conditions; most important of all, they provide samples which can be handled, examined, and tested. It is important that these samples should be as nearly as possible in the condition in which they exist in the ground, as disturbance and remoulding, especially in the presence of water, can alter the soil properties appreciably. For this reason many engineers prefer test-pits, since they provide a favourable opportunity of examining the soil strata "in situ." In addition, samples of soil can be readily obtained from a test-pit either by carefully removing a cube with a spade or by forcing in a sampling-tool (*Figs. 3* (*b*)) and digging it out. Even with test-pits, however, it is important to bear in mind that the soil conditions revealed in the summer may be very different from those which obtain when winter conditions prevail. As the depth of exploration increases, the cost of test-pits increases rapidly and unless soil conditions are very favourable the expense soon becomes prohibitive. Test-borings have the advantage that they can be readily carried out to the depths usually required, and it should perhaps be more generally known that with a method known as "undisturbed" sample boring it is possible to provide quite satisfactory samples for examination with most cohesive soils. (It is difficult to obtain undisturbed samples of sandy soils without considerable elaboration.) The method is rather more expensive than the normal test-boring, but the additional cost is usually amply compensated by the additional information which can be obtained from the "undisturbed" samples.

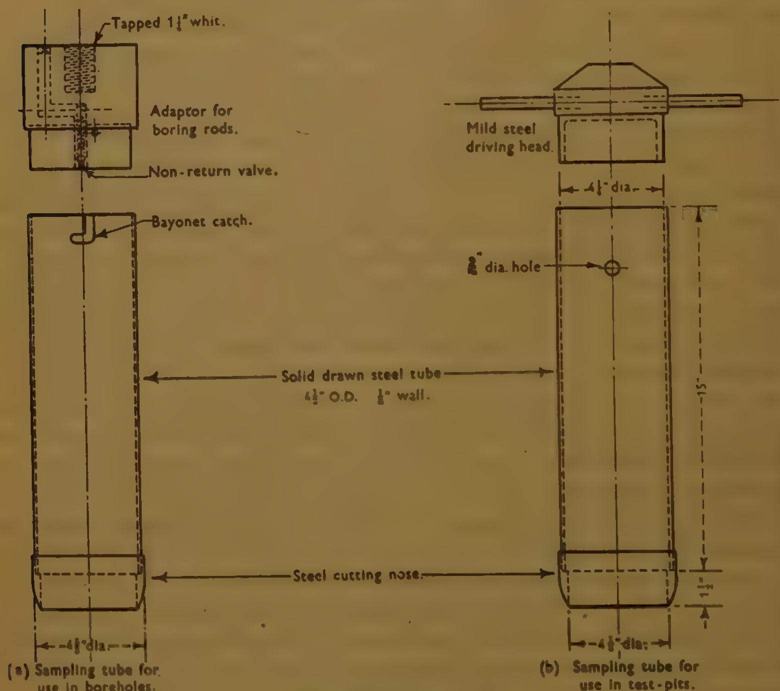
A comprehensive study entitled "The Present Status of the Art of Obtaining Undisturbed Samples of Soils" has recently been prepared by Mr. M. J. Hvorslev for the Committee on Sampling and Testing, Soil Mechanics and Foundations Division, of the American Society of Civil Engineers [1]. The following is a brief outline of the more important features of the sampling operations as applied to cohesive soils.

Ordinary well-boring apparatus is used and a hole at least 5 inches in diameter is sunk in the usual way with augers, chisels, etc., the hole being lined with steel liners, if necessary. As boring proceeds, small subsidiary samples, about ½ lb. in weight, consisting of auger parings, etc., are taken at frequent intervals, say over 2 feet, and stored in air-tight containers. These samples give valuable details of the variations in the type and consistency of the soil down the profile. Their use, however, precludes the artifice of pouring water down the borehole when boring is difficult.

[1] Supplement to Proceedings of the Purdue Conference on Soil Mechanics and its Applications, Purdue University, Lafayette, Indiana, July 1940.

When a depth is attained at which it is required to take an undisturbed sample, a sampling-tool (*Figs. 3* (*a*)) is fitted to the end of the boring-rods. The main features of the design of the sampling-tool may be briefly indicated as follows. The sampler consists of a cylindrical mild-steel tube with a fitting at the top for the boring-rods and a hardened steel cutting-nose. It provides for a sample 4½ inches in diameter, whilst a length of 15 inches supplies sufficient material for testing purposes in most cases.

Figs. 3



Soil Sampling Tubes.

The cutting-edge is slightly smaller in diameter than the interior of the cylinder; this reduces the friction between the soil sample and the tool to the minimum and is essential if compaction of the sample is to be eliminated. There is a slight " release " on the outside of the tool to cut down resistance on the outside, and a check valve is usually provided at the top to assist in retaining the sample during extraction. The cross section of the metal should be as small as possible consistent with strength requirements since a quantity of soil equivalent to the volume of the tool must be displaced when the sampler is forced into the ground. This feature of the design is usually expressed by the " area-ratio ", which represents roughly

the volume of the displaced soil in proportion to the volume of the sample [1]. The sampler in use at the Building Research Station has an area-ratio of about 20 per cent.

The sampling-tool is forced into the ground without rotation until it is full. Then the soil at the bottom is either cut by means of a wire-snare incorporated in the sampler or by rotating the tool to shear the soil at the cutting-nose. The sampler is then withdrawn and the sample is pushed out of the wider end with a screw device. The sample is coated with a liberal layer of molten paraffin wax, to preserve its moisture-content, and is packed into an airtight cylindrical tin with sand or sawdust to avoid damage in transit. The hole is then augered down a further distance and the operations are repeated to obtain an undisturbed sample at this new depth. In this way undisturbed samples for test and examination can be procured from all the important soil strata.

Mention should perhaps be made of a method now commonly used in the United States for preliminary investigations, in which continuous samples about 2 inches in diameter are obtained by driving into the soil a sampler made with thin-walled seamless steel tubing known as "Shelby tubing [2]." These samples are disturbed to a certain extent, but serve well for the laboratory investigation of "index properties."

A simple method of exploration has also been developed at the Building Research Station, for obtaining small samples $1\frac{1}{2}$ inch in diameter, which are substantially undisturbed. Although its application is limited in scope as regards the types of soil in which it can be used, and the depth of sampling, the method has proved valuable for the consideration of practical problems. This method is described on p. 48, *post*.

Indirect Methods. With the indirect methods of soil exploration the object is not to procure soil samples, but to obtain, by more rapid means, data relating to the general character and the approximate depth of dissimilar soil layers. These methods can be very useful for purposes of preliminary investigation, or, when used in conjunction with boreholes, to give a more detailed picture of the variation in thickness of soil strata over an extensive site. By themselves, however, they cannot provide the detailed information necessary for the final design of engineering structures. Sounding methods and geophysical methods are the two chief types of indirect method.

In the sounding method a rod, usually provided with a cone-shaped point, is forced into the ground and the resistance to penetration is measured as the depth of penetration increases. In recent work provision is made for the separate measurement of the point resistance and the frictional resistance of the sides of the rod. By this means a continuous

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1 Hvorslev, *loc. cit.*

2 H. A. Mohr, "Exploration of Soil Conditions and Sampling Operations." Harvard University Publication, Soil Mechanics Series No. 9.

record of the penetration resistance with depth can be obtained. These data can often give an idea of the general nature of the soil strata and furnish some indication of the required length of piles [1].

In the geophysical methods the depth of dissimilar soil layers is estimated from differences in electrical resistivity or in speed of propagation of artificially-produced vibrations, whether it is a single impulse of an explosion or continuous vibrations of known frequency. These methods are particularly useful for the rapid survey of large sites, especially when used in conjunction with boreholes [2, 3].

The Examination of Soil Samples.

The laboratory examination of soil samples is one of the principal objects of soil mechanics research, and important advances have been made in this field. The rough qualitative data which are the best that can be expected from visual and manual examination can now be supplemented by soil tests which give quantitative measurements of important soil characteristics. In addition simple tests, known as index-property tests, have been developed which, with the minimum of experimental work, serve to identify the soil type and to indicate its consistency.

The general procedure in the examination of a soil profile may be briefly described as follows. In the first place " index-property " tests are carried out both on the subsidiary samples already mentioned and on the undisturbed samples. The results, when plotted against depth, give an indication of the important changes in soil-type and consistency down the profile. A typical diagram is given in *Fig. 4*, which shows the variation with depth of the natural water-content, the liquid limit, and the plastic limit. The meaning and significance of these test constants has been described [4], but the following brief comments may be included. The liquid limit and plastic limit are both indicative of the type of soil; the higher the limit the more does the soil partake of a clay nature. The magnitude of the natural water-content in relation to the two limits is an indication of consistency; the nearer the natural water-content is to the liquid limit the softer is the soil. Results such as those in *Fig. 4* therefore give a quantitative indication of soil-type and consistency which serves to supplement the brief notes accompanying a boring-record. From such results it is possible to obtain a fairly accurate picture of the soil profile,

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[1] "The Predetermination of the Required Length and the Prediction of the Toe Resistance of Piles," Soil Mechanics Lab. Delft. Paper I, I, vol. I, Proc. Int. Conf. on Soil Mechanics and Foundation Engineering, Harvard, June 1936.

[2] E. R. Shephard, " Subsurface Exploration by Resistivity and Seismic Methods." *Public Roads*, vol. 16, p. 57. 1935.

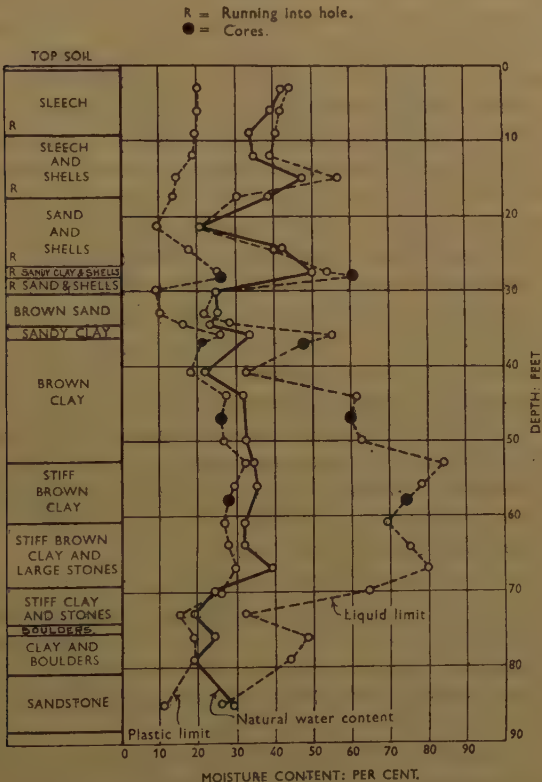
[3] H. M. Gell, " Subsurface Investigation by Electrical Methods." Journal Inst. C.E., vol. 9 (1937–38), Notices Section, p. [20], June 1938.

[4] L. F. Cooling and A. W. Skempton, " A Laboratory Study of London Clay." Journal Inst. C.E., vol. 17 (1941–42), p. 251.

to recognize the more important soil strata, and to select those undisturbed samples which should be subjected to the more elaborate mechanical tests.

In some problems, when a soil stratum is deep and fairly homogeneous, the results of the index property tests can be used in conjunction with the results of the mechanical tests to select a more accurate "average" value for the mechanical characteristics of the soil profile at a given point [1].

Fig. 4.



VARIATION OF INDEX PROPERTIES WITH DEPTH.

The two chief tests used for determining the mechanical characteristics of an undisturbed soil sample are the consolidation test and the shear test. The former is carried out in the oedometer [2], and the results give quantitative values for those characteristics, the compressibility and coefficient of consolidation, which control the gradual slow decrease in

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[1] Cooling and Skempton, *loc cit.*

[2] K. von Terzaghi, "*Erdbaumechanik,*" Deuticke, Vienna, 1925; or, more readily, L. F. Cooling and A. W. Skempton, "Some Experiments on the Consolidation of Clay," Journal Inst. C.E., vol. 16 (1940–41), p. 381. June 1941.

volume of a soil under the influence of an applied stress, and which permit an estimate to be made of the settlement movements of different parts of a structure [1]. Shear tests are carried out with the object of estimating the shear strength of the soil under the conditions at which it exists in the ground and the possible variation in shear strength which might result from changing weather conditions or from applied loads [2].

According to the nature of the problem, one or other of these tests becomes the more important. For instance, in many problems of foundation design the consideration which limits the permissible loading on a footing is that the differential settlement between various parts of the structure must be kept below a certain value, in order to avoid undue secondary stresses in the superstructure. For such problems the consolidation test combined with theoretical settlement analysis gives the important information. In other problems, for instance when the structure is very rigid and compact, the criterion is that the loading should be kept low enough to guard against excessive movements due to the deformation or failure of the soil in shear. In fact, in most foundation problems the first consideration of the engineer is to ensure an ample factor of safety against the possibility of failure of the soil in shear. Also, in problems relating to the stability of earth slopes and retaining walls, the shear strength of the soil is a controlling factor.

The study of the shearing resistance of soils is therefore important for most problems relating to foundations and earthworks, even if in some cases it is of value only as a means of preliminary investigation. For this reason a simple method of sampling and testing, based upon the measurement of the compression strength of the soil by tests in the field, has been developed at the Building Research Station.

It cannot be claimed that this method is applicable to sites in general: it is chiefly of value for sites where the foundation soils are predominantly cohesive, such as soft clays, silts, etc. However, these sites are among those which are most likely to give rise to foundation troubles, and since the method has proved of considerable practical value a brief description will be given.

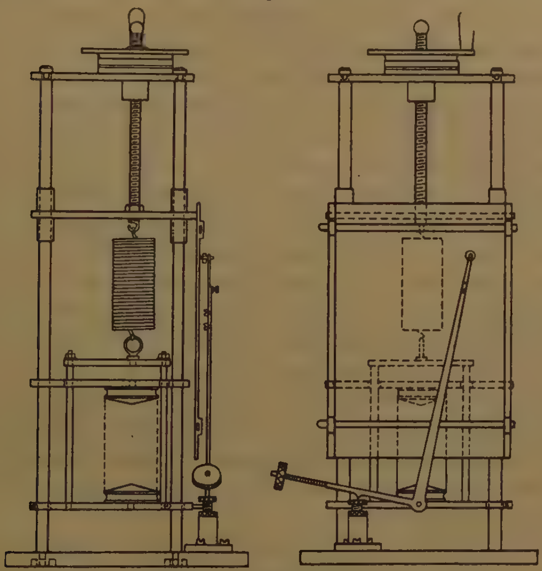
A 4-inch or 5-inch borehole is sunk with a post-hole auger, which can be fitted with extension-rods of $\frac{3}{4}$-inch gas-barrel. At frequent intervals of depth small samples are obtained by forcing into the ground at the bottom of the hole a brass sampling-tube 6 inches long and $1\frac{1}{2}$ inch in internal diameter, with a wall-thickness of $\frac{1}{16}$ inch. At one end is a cutting-edge formed by bevelling the tube on the outside; at the other end is a slot which engages with a bayonet catch on an adaptor which can be screwed to the boring rods. It has an "area-ratio" of 25 per cent. The sampler is forced into the ground without rotation by pressure applied at the top

[1] A. W. Skempton, "Settlement Analysis of Engineering Structures", *Engineering*, vol. 146, p. 403. 30 Sept. 1938.

[2] Report of the Building Research Board, 1937, p. 101.

of the rods, and when full it is rotated to shear the soil at the cutting-end and is then withdrawn. The sampler is then detached from the boring-rods, the bayonet catch making this operation very simple, and a cylindrical sample of soil is prepared and tested immediately in the portable compression apparatus illustrated in *Figs. 5*. This apparatus and details of the technique of testing have been described elsewhere [1]. Essentially the apparatus is a spring-loaded device which subjects a cylindrical sample of soil to an axial compressive stress. An autographic recording mechanism

Figs. 5.



UNRESTRAINED COMPRESSION APPARATUS.

draws the load-deformation curve, and by the use of a mask, consisting of constant stress-lines drawn on a transparent sheet, the compressive strength of the sample can be read off immediately. With cohesive soils the compression strength is twice the shear strength, and hence with such soils the compression apparatus gives a direct measure of the shear strength. In this way the variation of shear strength with depth can be accurately determined, and in suitable soils exploration has been carried to a depth of 25 feet below ground-level without difficulty. Another advantage is that the whole of the apparatus required can be carried in a small car. The small compression cylinders, after having been tested in the field, are placed in airtight bottles and transported to the laboratory, where they are subjected to simple tests to determine the weight per cubic foot and the " index properties."

[1] L. F. Cooling and H. Q. Golder, " A Portable Apparatus for Compression Tests on Clay Soils ", *Engineering*, vol. 140, p. 57, 19 Jan. 1940.

When deep boring is required, or when site conditions are not favourable (for example, when dense gravel seams have to be passed through or when the soil flows into the borehole), it is necessary to use the normal boring equipment and liners, etc., and in such cases it is probably best to obtain large undisturbed samples and examine them in the laboratory.

Application to Practical Stability Problems.

Site surveys, combined with a study of the properties of the various soils encountered, can yield information which is of value for supplementing the judgement of the engineer in the consideration of a number of practical problems. The use of this information takes the form of the application of broad general principles rather than the formulation of rules, and in general each problem requires to be considered on its merits. It is, therefore, thought that the best means of outlining the practical application of the methods of soil mechanics is to describe very briefly the technique employed and the results obtained in the investigation of a few of the practical problems which have been studied at the Building Research Station. As will be recalled from Prof. Terzaghi's "James Forrest" Lecture [1], soil mechanics is concerned with many types of problem; but since it is not possible to include the consideration of all these types in a short Paper, the investigations to be described will be limited to those relating to the stability of footings and earthworks.

In the analysis of stability problems the measurement of the shear strength by tests on samples in the field or in the laboratory has a direct application. This is particularly the case for sites on soft clays, since the analysis is here simplified by the fact that such soils tend to behave as purely cohesive materials.

Their shear strength increases only very slowly under the action of applied loads, and they can therefore be regarded as materials having a constant shear strength independent of applied pressure. With frictional soils, such as gravels and sands, the shear strength depends largely upon the magnitude of the applied load, and the analysis of stability problems is correspondingly more complicated. Those soils to which the simple method of site exploration particularly applies, and which often give rise to trouble, are therefore those which are amenable to comparatively simple theoretical treatment.

Broadly speaking, two lines of attack are available for the consideration of stability problems :—

(a) Methods based upon the analysis of the stability against complete failure of the soil in shear on a surface of assumed shape.

(b) Methods based upon elastic theory, in which a comparison is made between the shear stresses arising from the structural load and the shear strength of the soil as it exists in the ground.

[1] K. von Terzaghi, *loc. cit.*

Methods of the first group are based upon the principles of statical equilibrium, and in design problems the methods involve the finding by trial and error of the most dangerous surface of failure. They are of particular value in that they enable a quantitative estimate to be made of the factor of safety against complete shear failure. The methods of the second group are useful in giving a picture of the way in which failure is likely to take place, but they do not yield quantitative estimates of the factor of safety. The use of these methods in the analysis of stability problems will now be briefly described and illustrated with reference to the results obtained in investigations dealing with specific practical problems. Most of these investigations have been concerned with the analysis of failures, but it is, of course, from the examination of failures that the most valuable indications are given of the sort of information required for design purposes. They also relate to sites where the soils are predominantly cohesive. Each example is illustrated by a figure which includes the relevant information concerning the soils at each site. The type of soil is indicated by the results of "index-property" tests and the consistency by the results of shear tests.

(*a*) *"Ultimate" Bearing Capacity of Footings on Soft Clay.* For the calculation of the "ultimate" bearing capacity of a strip footing (that is, the intensity of loading which will cause failure of the soil in shear) various methods based on certain assumptions as to the shape of the failure surface have been developed by Fellenius [1], Krey [2], and others. For footings near the surface of a homogeneous layer of soft clay these methods lead to the following results for the "ultimate" bearing capacity (q_0) in terms of the shear strength (s) :—

$$\text{Fellenius}: \quad q_0 = 5{\cdot}5 \times s$$
$$\text{Krey}: \quad q_0 = 6{\cdot}0 \times s$$

Bell's method [3], applied to a cohesive soil ($\alpha = 0$) gives the result that $q_0 = 4 \times s$.

A method, based upon Prandtl's analysis in the theory of plasticity, the application of which to footings was suggested by Jürgenson [4], leads to the result that $q_0 = (\pi + 2) \times s = 5{\cdot}14s$. An extension of Prandtl's method to the case of a rigid circular area, which was developed by Hencky and quoted by Jürgenson, gives a value of $q_0 = 5{\cdot}64s$. In general, then, the theoretical methods suggest that the "ultimate" bearing capacity of

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[1] W. Fellenius, "*Jordstatiska Beräkningar för vertikal Belastning på Horisontal Mark under Antagande av Cirkulärcylindriska Glidytor*", *Teknisk Tidskrift*, vol. 59, No. 21, p. 57. 1929.

[2] Krey, "*Erddruck, Erdwiderstand und Tragfähigkeit des Baugrundes,*" p. 163. Berlin, 1926.

[3] A. L. Bell, "The Lateral Pressure and Resistance of Clay and the Supporting Power of Clay Foundations", Minutes of Proceedings Inst. C.E., vol. cxcix (1914–15, Part I), p. 233.

[4] L. Jürgenson, *loc. cit.*

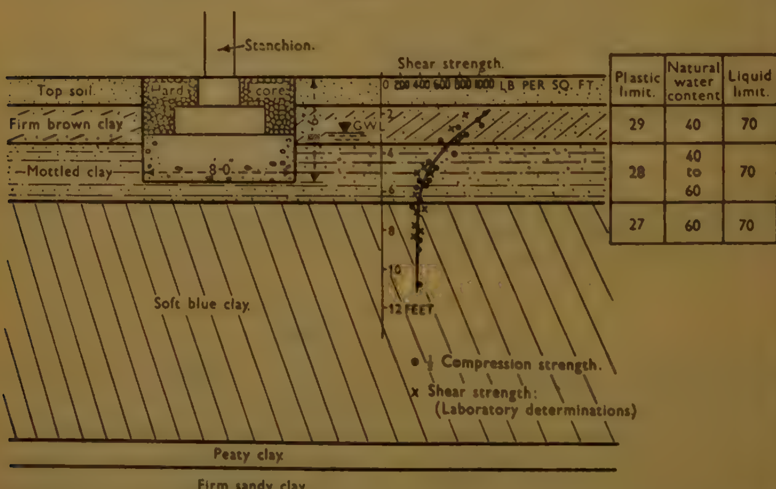
a strip footing near the surface of a soft clay is about five times the shear strength.

For footings located at a depth below the surface, extension of the theoretical methods is required, and much work remains to be done in this field. As a first approximation, it can be taken that the bearing capacity is increased by an amount νh, where ν denotes the unit weight of the soil and h the depth of the footing below the surface. This is conservative, since, apart from the weight effect, the restraint of the soil outside the footing is neglected. If then in the practical case the shear strength of the soil can be estimated from tests, the " ultimate " bearing capacity can be found.

Example I.

During the construction of a single-storey steel-framed building, one of the columns supporting a heavy roof settled about 10 inches in a week,

Fig. 6.



Footing Failure. Soil Profile Showing Variation in Shear Strength with Depth.

after the roof had been cast over it. The footing of the column was of mass concrete 8 feet by 9 feet in plan, and it was founded at a depth of 5 feet 6 inches below ground-level on what appeared to be reasonably good clay. From the progress of the casting of the roof it was possible to fix the failing load at about 1·4 ton per square foot.

Borings and field tests were carried out on the site by the method described on p. 48, *ante*, and took about 4 days to complete. The results are indicated in *Fig. 6*, which shows the soil profile and the variation in shear strength with depth. By taking appropriate average values of the

shear strength and using these in the theoretical methods of analysis, the following results were obtained. By the method based on the work of Prandtl the theoretical "ultimate" bearing capacity was calculated to be 1·2 ton per square foot, whilst the method of Fellenius gave 1·4 ton per square foot.

The substantial agreement between the estimated actual failing load and the theoretical failing load given by this example constitutes a valuable check on the validity of the theoretical methods of analysis and the method of site exploration.

A full description of this investigation and a more detailed discussion on the methods of analysis will be included in a Paper by Mr. A. W. Skempton [1].

(*b*) *Allowable Bearing Pressure of Footings on Soft Clay.* The analysis quoted in the previous section refers to the condition of complete failure of the soil in shear; consequently an ample factor of safety on these values must be allowed when questions of design are under consideration. For purposes of design the approach based upon the theory of elasticity is probably preferable. When the maximum shearing stress becomes equal to the shear strength of the soil at a point, a plastic zone tends to develop, and it therefore appears reasonable to adopt as a criterion in design the suggestion put forward by Carothers [2] that the footing load should be so limited that at no point does the shear stress exceed the shear strength of the soil. The maximum shear stress set up in the soil by a strip or circular load of intensity p is p/π, and therefore, if plastic yield is to be prevented, p should not exceed π times the shear strength. A criterion for design purposes for a homogeneous soil may then be expressed by the equation:

$$p_1 = p - \nu h = \pi s$$

where p_1 denotes the net intensity of pressure which is obtained by subtracting from p (the gross pressure) a pressure νh which is the equivalent of the soil excavated to founding-level.

In the practical case it is necessary to compare the stress contours with the strength of the soil at various depths, to see that nowhere is the shear strength exceeded.

Example II.

A building, 125 feet long and 62 feet wide, which was supported on a honeycombed concrete raft located at about 3 feet below ground-level, had shown a continuous gradual settlement amounting to about 10 inches in 12 years, and movement was still continuing. The net pressure imposed by the building (that is, the total pressure of 0·37 ton per square foot less 0·12 ton per square foot due to the soil excavated to founding-level) was

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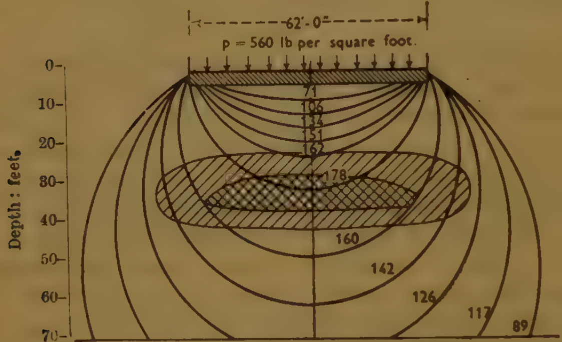
[1] A. W. Skempton, "An Investigation of the Bearing Capacity of a Soft Clay Soil" (to be published in a subsequent issue of the Journal).

[2] S. D. Carothers, "Test Loads on Foundations as Affected by Scale of Tested Area", Proc. of Internat. Math. Congress, Toronto, 1924, vol. ii, p. 527.

only about 0·25 ton per square foot, but the underlying foundation soil was very soft. The soil strata, which were fairly uniform over the site, consisted of 5 feet of hard clay resting on soft mud which became softer with increase in depth until at 40 feet below ground-surface a thick stratum of harder clay was encountered. Samples of soil from various depths were sent to the laboratory for test. Theoretical analysis indicated

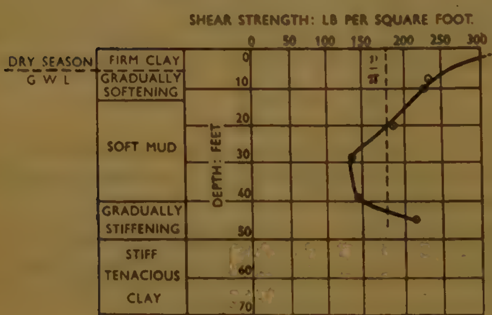
Figs. 7.



(a) VARIATION OF SHEAR STRESS WITH DEPTH.

0-30 lb. per square foot overstress.

> 30 lb. per square foot overstress.



Plastic limit.	Natural water content	Liquid limit.
31	32–80	70–90
30 to 36	65 to 85	75 to 100
25–31	27–40	75–100
25–30	26–30	55–80

(b) VARIATION OF SHEAR STRENGTH WITH DEPTH

OVERSTRESSED ZONE IN EXAMPLE II.

that the whole of the settlement could not be accounted for by consolidation of the soil, and the possibility of plastic yield of the soft undersoil was therefore investigated. The variation of the shear strength with depth is shown in *Figs.* 7 (*b*), and when this is compared with the theoretical shear stresses set up by the structural load, it is found that within the zone indicated in *Figs.* 7 (*a*) the soil is overstressed in shear. This zone is supported by the stronger clay above it, which prevents complete failure in shear, but a certain degree of plastic yield takes place which contributes to the observed settlements mentioned above.

(*c*) *Loading Tests on Small Areas.* The traditional method of estimating the bearing capacity of a soil is to carry out loading tests on areas a few square feet in size. If the foundation soil stratum is fairly uniform and deep, the results may furnish a valuable indication, but in some cases they may be misleading, for the following reasons. The results of a test on a small footing, of breadth say $2b$, are mainly influenced by the nature of the soil just below the footing to a depth $3b$. The load of the structure will, however, influence the soil to a much greater depth, and the zone of influence may embrace soil strata very different from those encountered near the surface. In such cases it is very difficult (if not impossible) to utilize the information obtained from a loading test.

On the other hand, the methods described in (*a*) and (*b*) above can be readily adapted to take into account complex site conditions, and are therefore much more comprehensive in their applicability. In addition, they are easier to carry out and enable the variation over a site to be studied in much more detail than would be possible with loading tests.

However, a loading test gives results which agree with those obtained from an analysis of the test footing on the basis of elastic theory. Some interesting experimental evidence confirming this has been published in a recent Paper [1] describing a series of field loading tests which were carefully carried out on areas ranging up to 16 square feet. In each test, by suitable choice of load increments, a " yield-point " load was estimated from the load-settlement curve. Shear tests were also carried out on the various soils concerned and a significant correlation was found between the shear strength and the " yield-load " ; in general the " yield-load " was found to be of the order of three times the shear strength.

(*d*) *Stability of Clay Slopes.* The analysis of the stability of a clay slope requires a knowledge of the shearing resistance both in the body of the bank and in the foundation soil below it. Banks of cohesive soil usually fail by shear along a curved surface, and in many cases the trace of this surface can be represented with sufficient accuracy by the arc of a circle. The usual method of analysis is to consider the statical equilibrium of such a wedge by comparing the moment of the disturbing forces about the centre with the moment of the resisting forces. The method was outlined by Professor Terzaghi in his " James Forrest " Lecture of 1939 [2]. If the average shearing resistance s around the surface of failure is estimated, the factor of safety of the slope can be determined by taking a number of possible circles and finding the most dangerous one by trial and error. In some practical cases, however, when the soils are far from homogeneous, the surface of failure tends to depart from the circular form and to pass through the weakest layers. In problems of this kind measurements to show the

[1] T. R. Dames, " Practical Shear Tests for Foundation Design ", *Civil Engineering* (New York), vol. 10, No. 12, p. 783. Dec. 1940.

[2] *Loc. cit.*

degree of variation in shear strength are especially important, and are helpful in indicating whether a modification of the usual method of analysis is called for.

Example III.

A failure occurred during the construction of an earth dam, a section of which is shown in *Fig. 8*, when the weather conditions were dry. The dam had a central core of puddle clay and was built on a thin stratum of soft alluvial clay. An indication of the general shape of the failure surface was obtained by inspection from a trench cut into the bank, and it was found that it could be represented fairly accurately by two circular arcs of different radii. The shear strengths of the various soils were measured and an analysis of the failure was made embodying the following modifications to the usual method. An assumption was made that the force P between the two sections A and B (*Fig. 8*) acted at the lower third in a direction perpendicular to the common radius. Other reasonable assumptions were also tried, and it was found that the result was not influenced to any large extent by the nature of the assumption. Then by considering the equilibrium of the first section about 0_1, the magnitude of this force was estimated. With the position and magnitude of the force given the equilibrium of the second section about 0_2 was calculated. The factor of safety determined by this method was 1·1, which is sufficiently near to 1 to confirm the validity of the method used.

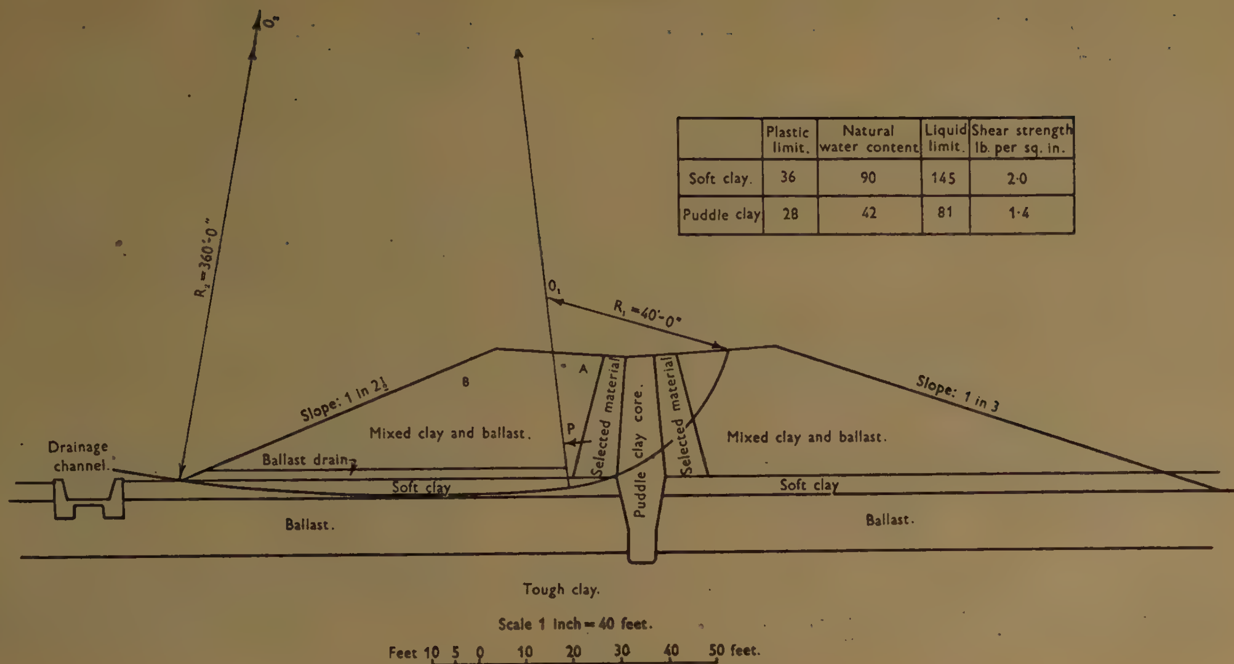
(*e*) *The Yield of a Soft Foundation Layer under the Weight of a Bank or Fill.* Although the factor of safety against complete failure of a bank or fill on a soft foundation layer can be estimated by the method described above, there may be cases where substantial yield is likely to take place short of complete failure. Such problems are best considered by methods based on the theory of elasticity, and Carothers [1] and Jürgenson [2] have derived expressions for the maximum shear stresses in the foundation soil with various types of applied external loads. The general principles of the application of the method are illustrated by Example IV.

Example IV.

A single-storey building had been constructed below ground-level in a cutting. At the end of the main building was a subsidiary structure which came to within 5 feet of the toe of the cutting. The cutting was 16 feet deep, with slopes of 1 in 1½ in a hard brown clay, and the whole building was founded on good firm clay (*Fig. 9*). Following a period of heavy rain, the floor of the subsidiary structure cracked badly. Levels indicated that the centre of the floor had risen about 3½ inches relative to the end wall of the main building, which had not moved to any appreciable extent.

[1] *Loc. cit.* [2] *Loc. cit.*

Fig. 8.



	Plastic limit.	Natural water content	Liquid limit.	Shear strength lb. per sq. in.
Soft clay.	36	90	145	2·0
Puddle clay	28	42	81	1·4

STABILITY ANALYSIS OF EXAMPLE III.

The movement was effectively stopped by cutting back the side slopes of the cutting. A visit was paid to the site, and in about 4 days, using the simple method of site exploration, information was obtained which enabled the section in *Fig. 9* to be prepared. The survey showed that although the slope of the cutting and the soil immediately beneath the structure consisted of hard fissured clay, a stratum of very soft clay existed lower down. The theoretical calculation of the maximum shear stresses was based upon the consideration of a system of " terrace loading " for which figures have been conveniently tabulated by Jürgenson [1]. The calculated theoretical stresses are given in *Fig. 10*, and it will be seen that the balancing weight of the structure has been included by reducing the effective height of the bank. Comparison between these stress contours and the strength contours (*Fig. 9*) shows a fairly extensive zone where the soil is substantially overstressed (*Fig. 10*).

A similar analysis, made with the bank cut back, indicated that the overstressed zone was then considerably reduced both as regards the degree of overstressing and the extent of the zone.

An analysis by the method described in Example III was also used, and it was found that the factor of safety of the bank in its original position was about 1·3, and that when it was cut back the factor of safety increased to about 1·6. In problems where deformations must be prevented it is, therefore, necessary to use an adequate factor of safety against complete failure, just as in the case of footings.

(*f*) *Disposition of a Spoil Bank on Soft Clay.*

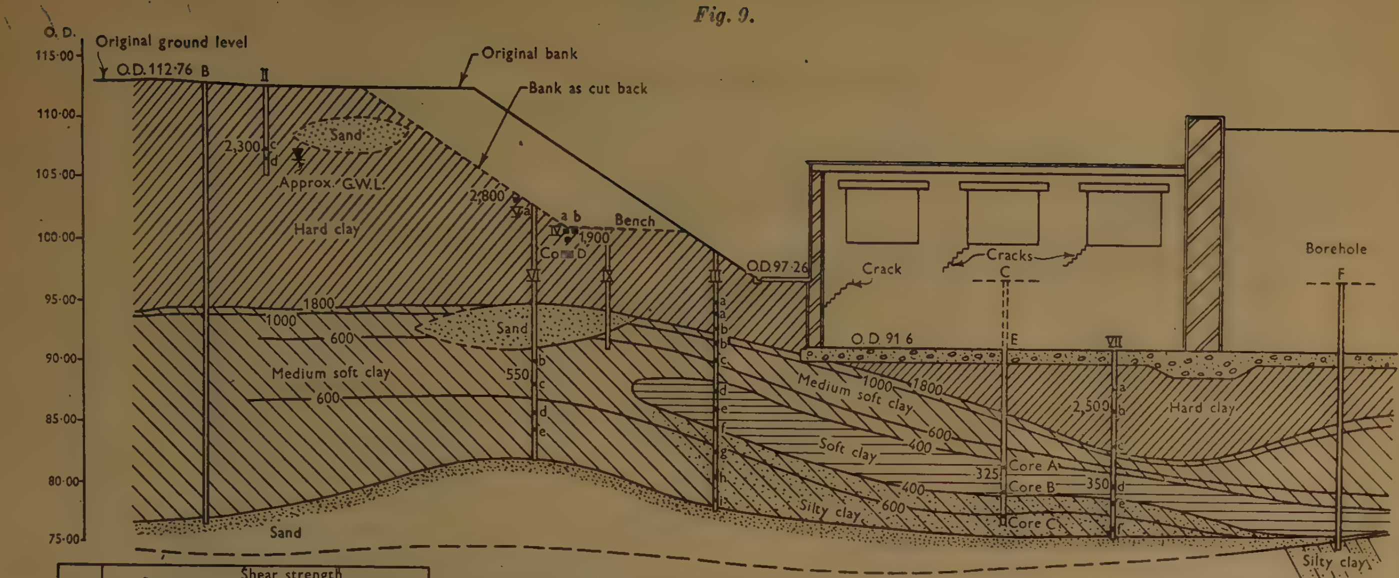
Example V.

This example is included because it was concerned with a question of design. The problem related to the construction of a drainage-channel which was to be cut through beds of very soft alluvial clay, and the disposition of the spoil therefrom. Site exploration showed that for a depth of 32 feet the soil strata consisted of very soft clays, but that below this depth the strata tended to become much stiffer. Observations made during the early construction period, together with a theoretical analysis based upon measured values of the shear strength of the clay, indicated that a cut 16 feet deep would stand with a reasonable factor of safety with a slope of 1 in 4.

In order to obtain some idea of a suitable slope for the spoil bank, the usual method of analysis was extended and applied after the manner indicated in *Fig. 11*. It was assumed that the most probable slip surface would be limited in depth by the comparatively hard stratum at 36 feet below ground-level. Then with the proviso that the factor of safety should be kept the same as that for the cut alone (without spoil bank) the way in which the height of the spoil bank could be increased was

[1] *Loc. cit.*

Fig. 9.



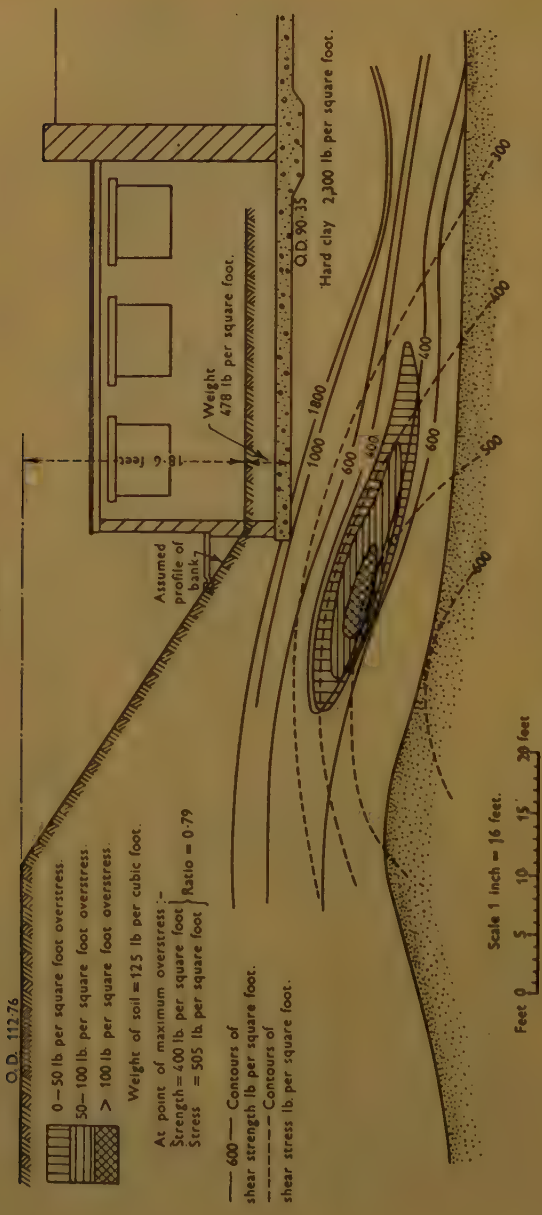
	Soil type	Shear strength lb./ft.²	Liquid limit
	Hard clay	1800 – 2000	50 – 60
	Medium soft clay	400 – 1000	40 – 50
	Soft clay	300 – 400	40 – 50
	Silty clay	400 – 1000	25 – 40

—600—Contours of shear strength lb per square foot.
2,500 Spot values of shear strength lb per square foot.
Positions of Test specimens

Scale 1 inch = 16 feet
Feet 0 5 10 15 20 feet.

Soil Conditions at Site of Example IV.

Fig. 10.

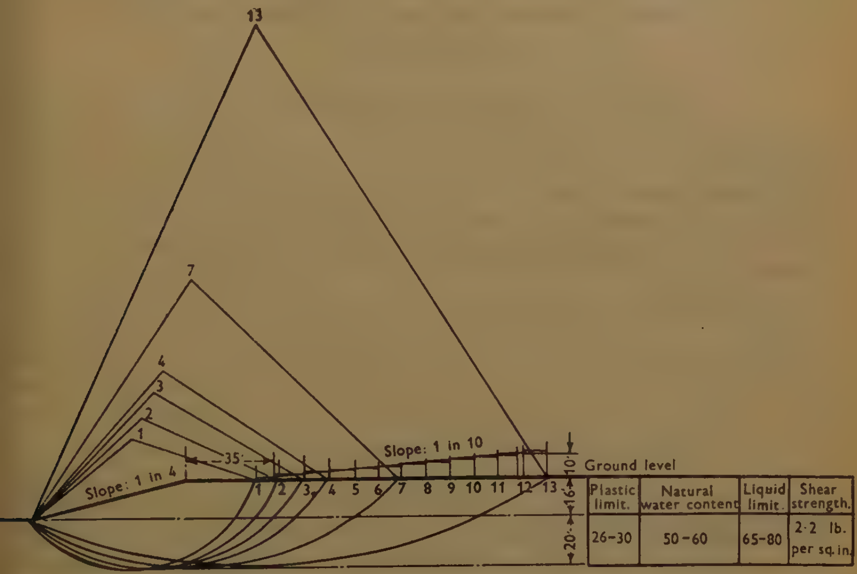


Overstressed Zone in Example IV.

calculated. The results of the analysis suggested that the berm should be made at least 35 feet wide and that the slope of the spoil bank should be limited to 1 in 10.

Subsequent reports of the behaviour of the cut and spoil bank indicated that when this procedure was followed no trouble had ensued, but that attempts to place the spoil bank nearer the cut had led to minor failures.

Fig. 11.



THEORETICAL SHAPE OF SPOIL BANK.

ACKNOWLEDGEMENTS.

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(Paper No. 5297.)

"Soil Mechanics in Road and Aerodrome Construction." * †

By Alfred Herbert Dorlencourt Markwick, M.Sc., Assoc. M. Inst. C.E.

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Introduction.

The soil problems in both road and aerodrome construction are essentially similar, since in both cases the basic object is to provide a surface that will carry wheeled traffic at all seasons of the year. Whilst other branches of soil mechanics are usually concerned with the behaviour of the soil at considerable depths below the surface, in roads and runways the top 2 or 3 feet are of greatest importance. Seasonal variations in the moisture-content of this layer of soil may result in considerable changes in its mechanical properties, so that direct measurement of these properties at any given time may not give an adequate idea of the probable behaviour of the soil. Therefore it is necessary to infer this behaviour from the nature of the soil and the conditions on the site.

Important differences between road and aerodrome problems arise, however, from the differing nature of the wheel loads. Owing to the smaller pressures applied by certain types of aircraft, and to the fact that repetition of the load in the same track is infrequent, it is possible to use grass surfacings alone in some cases and in many others to use surfacings that would be inadequate for the more intense conditions of road wear.

† Correspondence on this Paper can be accepted until the 15 July, 1942, and will be published in the Institution Journal for October 1942. Sec. Inst. C.E.

At the other extreme, the heavy wheel loads applied by the largest aircraft now in use stress the soil more severely and to greater depth than do wheel loads on roads.

The problems involving soil in road and aerodrome construction may be roughly classified as follows :—

(*a*) *Foundations :* bearing capacity and behaviour of foundations under traffic ; movements due to seasonal and other changes in the moisture-content of soil ; drainage ; behaviour of soil under concrete slabs and other forms of surfacing ; construction on peat foundations and on swampy and soft ground ; frost damage.

(*b*) *Earthworks :* selection of filling materials ; control of compaction in earthwork construction ; embankment settlements and road behaviour ; bulking of earthwork ; stability of cuttings and embankments ; landslides ; trench reinstatement ; backfilling behind retaining walls and bridge abutments ; earthwork construction methods and machinery.

(*c*) *Soil stabilization.*

(*d*) *Retaining walls.*

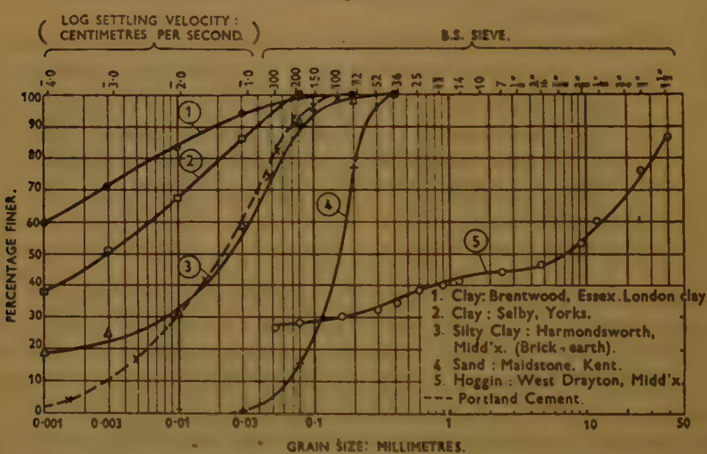
Within the compass of a single Paper it is only possible to cover a few aspects of the work. A brief survey will first be given of the nature of soil and its mechanical properties. In every application of the work it is essential to sample the soils present on the site, and a detailed account is therefore given of the methods used for site exploration (" soil surveys "). An analysis is made of the bearing capacity of soils under both stationary and moving aeroplane wheels. On the important subject of drainage very little information appears to be available, and reference is therefore confined mainly to modern American practice in which an attempt has been made to relate the drainage provided to the nature of the soil. Problems involved in embankment construction are also reviewed.

Nature of Soil.

Soil consists mainly of a mixture of mineral particles and water. In the engineering sense, the term " soil " is applied to all superficial deposits composed of mineral particles of varying size derived from the natural disintegration of the rock crust of the earth. It therefore includes a wide range of materials, from gravel and shingle on the one hand to plastic clay on the other. When organic material is associated with the mineral particles, as in peat, it may have a considerable effect upon the nature and behaviour of the soil. In general the type of soil is determined by the size, shape, and nature of the soil particles, but the properties—in particular the consistence—of any given soil depend almost entirely upon its moisture-content.

The wide range of particle-size distribution found in soils is indicated in *Fig. 1*, where the logarithmic scale covers a range of particle-size from 50,000 to 1. Even this does not include the lower limit of size, which is probably below $0{\cdot}1\mu$[1]. Although very fine, these particles are by no means of molecular size, for $0{\cdot}1\mu$ is equal to 1,000Å, whilst the spacing of the crystal lattice of montmorillonite, one of the common minerals occurring in clay, is of the order of 10Å. The dotted curve in *Fig. 1*, showing the mechanical analysis of a typical Portland cement, gives an idea of the fineness of clays in terms which can be readily appreciated.

Fig. 1.



MECHANICAL ANALYSIS CURVES FOR TYPICAL SOILS.

SOIL CLASSIFICATION TESTS.

Classification tests for soils have been widely used during the past 10 years, and were standardized in 1939 by the American Society for Testing Materials. They meet the need for a more definite means of identifying soils than is possible by visual inspection, and, as already stated, have to take the place in many road and aerodrome problems of more extensive mechanical testing.

The tests are of two types, namely, (1) mechanical analysis, a combination of sieving and sedimentation used to determine the size-distribution of the soil particles; (2) index tests (confined to that portion of the soil passing a 36-mesh British Standard sieve) by means of which the type of soil is inferred from the moisture-content at various standard consistences.

No attempt will be made here to describe these tests in detail. Briefly, the "liquid limit" of a soil is that moisture-content at which the soil is

[1] 1μ or 1 micron = 0·001 millimetre.

sufficiently fluid to flow a specified amount when lightly jarred on standard apparatus, and the "plastic limit" is the lowest moisture-content at which a thread of soil can be rolled down without breaking until it is only $\frac{1}{8}$ inch in diameter. The index properties of the soils whose mechanical analysis is shown in *Fig. 1* are as follows:—

Reference No. of soil (*Fig. 1*).	Plastic limit: per cent.	Liquid limit: per cent.	Plasticity index (= liquid limit − plastic limit): per cent.	Clay-content ($< 2\mu$): per cent.
1	29	92	63	66
2	24	53	29	45
3	19	34	15	20

The test values increase roughly in proportion to the "clay"-content of the soil. Whilst, on the average, this relationship has been shown to be true for a large number of British soils, it is seriously affected by the presence of organic or micaceous particles, and probably also by changes in the mineralogical composition of the clays.

The liquid and plastic limits of a soil mark widely spaced limits of soil consistence within which the natural moisture-content is usually found. The consistence of a soil at its natural moisture-content can thus be conveniently expressed by its "liquidity index", defined as follows:—

$$\text{Liquidity index} = \frac{\text{amount by which the natural moisture-content exceeds the plastic limit}}{\text{amount by which the liquid limit exceeds the plastic limit}} \times 100.$$

The results of these and other classification tests on soils have been correlated with the behaviour of the soil as a road foundation. The best known classification of this type is that of the United States Public Roads Administration, which is described in textbooks on Soil Mechanics[1, 2].† Soil classification is also useful in determining the general suitability of soil for stabilization [3]; reference is made later in this Paper to its application in the design of subsoil drainage on aerodromes, and to its bearing on the value of soil for use as filling in earthwork construction.

The mechanical properties of soils have been adequately described elsewhere, but it is of importance to recall that the shear strength of the soil may be approximately represented by Coulomb's equation:

$$s = c + p \tan \phi \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where c denotes the cohesive strength of the soil, ϕ the angle of internal friction of the soil, and p the normal pressure on the surface under shear.

For a given soil at given moisture-content, c is taken as a constant and

† The references are to the bibliography on pp. 86, 87, *post*.

may have any value from zero for non-cohesive soils (for example sand) up to 100 lb. per square inch for a particularly hard, dry clay: the cohesive strength of a stiff clay would be about 10 lb. per square inch. The value of ϕ varies with the type of soil and diminishes as the grain-size becomes finer. For non-cohesive soils (sands) it may vary from about 33 degrees to more than 40 degrees when they are densely compacted. The effective angle of friction in cohesive soils is small and may be anything from zero to about 15 degrees. Soils with a plasticity-index exceeding, say, 25 may be classed as cohesive soils, and soils with plasticity indexes from 0 to 10 as granular soils. Soils with plasticity indexes ranging from 10 to 25 are intermediate in both character and mechanical properties.

Soil Surveys for Roads and Aerodromes.

A soil survey involves sampling soil types in the field and recording their distribution and characteristic properties. Laboratory tests are made on representative samples of the soil to determine their nature and properties, and water-table levels are recorded to give information on the natural drainage of the site.

On account of the shallow depths to which borings are made, lining tubes are not usually required. Hard strata, which generally indicate a satisfactory foundation, need not, as a rule, be penetrated. It is therefore possible to employ very simple boring equipment. At the Road Research Laboratory, post-hole augers are extensively used in making borings. *Figs. 2* (*a*) and (*b*) illustrate the cutting-tools of the 5-inch and 4-inch augers used in ordinary soil not containing stones. Under favourable conditions borings have been made with augers of this type to a depth of 25 feet by adding extension-pipes. In reasonably easy ground, two men can take 50–60 feet of borings per day. Difficulties are, however, experienced in driving through gravel, and although a tool for this purpose is available (*Figs. 2* (*c*)), progress is rather slow. The soil augers described, which are similar to those favoured in the United States[4], appear to be superseding the corkscrew type of smaller diameter which is sometimes recommended[5].

Before deciding upon the position of the borings it is desirable to walk over the site to obtain a general idea of soil conditions. Next the centre-lines are set out, approximate methods usually being sufficient, and marked pegs are driven at chainages where borings are to be made. For a road, borings are usually made at 300-foot intervals along the centre-line, but when wide variations in the levels or in the type of soil are met, borings are spaced at closer intervals. On aerodrome sites, borings are usually made along the centre-lines of the runways, but if the lay-out is not known, borings are taken over a grid of points covering the entire site.

A profile survey is then made, intermediate levels being recorded to the nearest 0·1 foot. Finally, borings are made, and typical samples of

(*a*) 5-in. Auger. (*b*) 4-in. Auger. (*c*) Gravel Auger.

TYPICAL BORING TOOLS USED IN SOIL SURVEYS.

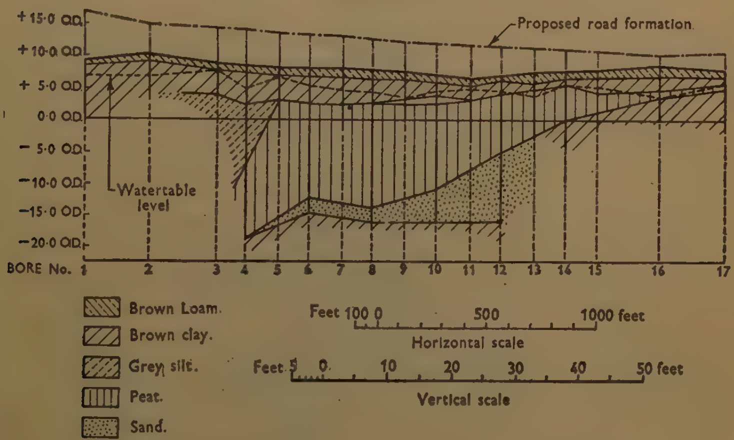
Fig. 5.

EQUIPMENT FOR TAKING UNDISTURBED SAMPLES OF SOIL.

soil are taken from various depths. These are placed in numbered lever-lid tins which are forwarded to the laboratory. The depth of the water-table is measured from the ground-surface, but it is usually desirable to wait some hours after a boring has first been made, to ensure that the water has risen to the true water-table level. Records are made of each boring.

From the information obtained in the field, a " soil profile " is drawn, consisting of a section along the centre-line indicating the various strata. In the case illustrated in *Fig. 3*, the borings disclosed the unsuspected presence of a thick bed of peat which could not be avoided by any practicable deviations. The maximum depth of borings in this case was 25 feet,

Fig. 3.



Typical Soil Profile Showing Concealed Bed of Peat.

but it was possible to attain this depth with hand tools only by sinking the holes with the utmost speed. The positions from which samples have been taken are indicated on the draft profile survey, and typical samples are then selected, on which classification tests are made. In general moisture-contents are also recorded for all the samples taken.

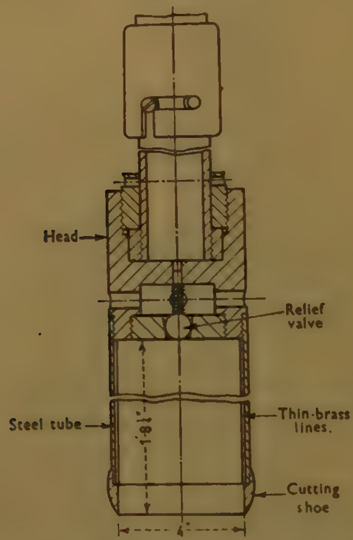
The index tests give no direct measure of the mechanical properties of the soil : these must be determined on " undisturbed " samples, since they depend upon the closeness of packing of the soil as well as upon its nature and moisture-content. The methods of sampling and testing used are essentially similar to those employed in the study of structural foundations, but the equipment used at the Road Research Laboratory embodies one or two interesting features. The undisturbed sampling-tool (*Fig. 4*) is fitted with a sheet-brass liner, into which the sample is pressed. The liner permits the sample to be readily removed from the tool and gives additional protection in transit to the laboratory, where it is removed by

unsoldering tacks along a seam. The tool is forced into the ground under direct pressure from a 5-ton double-acting jack mounted on a frame that can be fitted to the back of a lorry (*Fig. 5*).

Loading Conditions and the Bearing Capacity of Soil under Aeroplane Tires.

Since it is frequently difficult to get grass surfaces to withstand the intensity of traffic and the heavy wheel loads now imposed, it is becoming general practice to construct paved runways 150 feet or more in width

Fig. 4.



Sampling Tool for Obtaining Undisturbed Soil Samples.

and from 3,000 feet to 5,000 feet long in several directions, to permit aircraft to land at all seasons (*Fig. 7*). Both the bearing capacity of the soil alone and the bearing capacity of paved runways must therefore be considered.

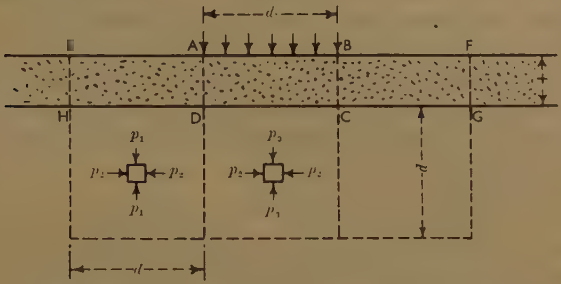
The maximum wheel loads of aeroplanes are much higher than those of road vehicles. Present British practice cannot be quoted [1], but according to the United States Civil Aeronautics Authority (C.A.A.)[6] 100,000 lb. should be regarded as the maximum static gross aircraft weight for present requirements, and 300,000 lb. for probable developments within the next 10 years. Already one of the largest civil aircraft, the Douglas DC–4, weighs about 63,000 lb. The static wheel loads will be about half this value, but to this must be added impact loads when the aircraft fails to

[1] Since the Paper was written, it has been announced that the weight of the Stirling bomber exceeds 30 tons (*The Times*, 27 Jan. 1942).

make a perfect landing. It has been suggested that for the Douglas machine these loads may approach four times the static load[7].

The carcass of an aeroplane tire is relatively light and flexible, and it is probably sufficient to regard the inflation-pressure in the tire as transferred directly to the soil and to neglect the additional stresses produced by the

Fig. 6.



BEARING CAPACITY OF A FLEXIBLE SURFACING: HOUSEL'S METHOD.

Fig. 7.

CONTOURS OF WATER-TABLE AND SHORE LINE.

deformation of the tire carcass. Present-day tire-pressures are of the order of 50 lb. per square inch, but the C.A.A. estimates that with increased loads the probable range of tire-pressures in the future will be from 50 lb. to 85 lb. per square inch. Tire contact-areas will therefore be of the order of from 600 square inches to 1,000 square inches for a 100,000-lb. aeroplane, and the major axis of the tire-ellipse, assuming, say, a 1·7 : 1 ratio between major and minor axes, will be 3–4 feet.

In the absence of experimental data, the only way to study the bearing capacity of the soil under aeroplane tires is by a combination of analysis with practical experience. The simplest and most severe case—that of a stationary wheel—will be considered first. The problem will be further simplified by assuming that the contact-area is circular. The stresses produced by such a load on a semi-infinite elastic body have been determined[8], and the maximum shear strength has been shown to occur at a depth of about two-thirds the radius of the loaded area. This means that even soil 2–3 feet from the surface is severely stressed.

In Appendix I, an approximate analysis has been made of the bearing capacity of a loaded circular area resting on homogeneous soil. In the particular case of a circular load on a purely cohesive soil, the bearing capacity q is given by the formula :

$$q = 5c + a\rho/6 \qquad \text{(2)}$$

where ρ denotes the weight of soil per cubic foot, and a the radius of the area of contact.

The substitution of numerical values shows that the last term is negligible, indicating that the bearing capacity is governed almost entirely by the cohesive strength of the soil and is independent of the wheel load. On a granular soil, however, the unit bearing capacity increases with increase in wheel load, as shown in Appendix II.

The principal conclusions drawn from the analyses in the Appendixes are as follows :—

(1) The pressure on the ground is equal to the inflation-pressure of the aeroplane tire.

(2) On purely cohesive soils, the ground will be stable provided that the cohesive strength of the soil exceeds about one-fifth of the tire-pressure.

(3) On a purely granular soil, for example, sand, the allowable bearing pressure increases with tire-size in proportion to (wheel-load)$_{\frac{1}{2}}$, and hence in the ratio of the linear dimensions of the area of contact.

(4) On a purely granular soil, the bearing capacity increases rapidly with increasing angles of friction : hence densely-compacted sand has a higher bearing capacity than loosely-compacted sand.

(5) On a purely granular soil, surcharge due to the dead weight alone of any surfacing permits a considerable increase in the allowable bearing pressure.

(6) The allowable bearing pressure on a purely granular soil is considerably reduced when the water-table approaches the surface.

(7) When a soil possesses both cohesive strength and internal friction, the bearing capacity increases considerably with increasing angles of internal friction.

Under a moving tire several additional factors arise, such as impact loads and the effects of transient loading on the shear strength of the soil. Tractive resistance caused by rutting increases the wheel loads. Impact loads also increase the contact-areas considerably, but since the bearing pressures will again be almost the same as the tire-inflation pressure, it follows from the previous paragraph that increased wheel loads should not reduce the bearing capacity of a homogeneous soil. This is borne out by experience, since the depth of rut produced when a heavy aeroplane lands on soft ground increases, as the machine slows down, very roughly inversely as some function of the velocity. The inertia of the great mass of soil that must be displaced when a tire sinks into the ground limits the deformation possible in the short period during which the tire is in contact. The analysis given in Appendix I has been applied to the case of contact for short periods of time, and calculations indicate that the deformations under the tire may be quite small even when the soil is far from capable of supporting the wheel loads under static conditions. The case is analogous to that of a skater on thin ice, which will support him if he is moving sufficiently rapidly, but will break under his weight if he stops.

Pressures in and Beneath Runways.

Runway surfacings are of two main types, namely, rigid surfacings having appreciable transverse strength, such as concrete, and flexible surfacings such as tar macadam on pitched foundations, and sand asphalt. Test data quoted by Hogentogler and Terzaghi[9] show that the mean intensity of pressure on the ground beneath a flexible surfacing may attain eight times the mean pressure under a concrete surfacing. Flexible surfacings are, however, less affected by settlements than are rigid slabs.

The problem of concrete runway design is essentially the same as that of concrete roads, except that the wheel loads and tire contact-areas are larger. Westergaard[10] has recently shown that the methods of calculating the stresses in concrete road slabs can be applied to runways with only slight modifications. The basic assumption of this theory, which is confirmed by experiment, is that the soil beneath the concrete behaves elastically. The intensity of pressure required to produce unit deflexion is termed the " modulus of subgrade reaction ", and the first step in calculating the stresses imposed by wheel loads is to determine a suitable value for this modulus.

It has been shown[11], however, that the traffic stresses depend mainly upon the applied load and the thickness of the surfacing, and that the effect of changes in the value of the modulus of subgrade reaction is comparatively small. Knowledge of the precise value of this modulus is therefore unnecessary, and in practice it is probably sufficient to vary the thickness of the concrete according as the soil is a good or indifferent foundation material, and to distinguish between made and unmade ground.

Thus the problem of the stresses in concrete surfacings is not primarily one of soil mechanics. This question is also discussed in the Manual of the U.S. Corps of Engineers [7].

The theory of flexible surfacings is by no means complete, and for lack of a better method it is commonly assumed that a flexible surfacing spreads the vertical stress through the frustum of a 45-degree cone and that the pressure is distributed uniformly over the base of this cone. If the tire load W be assumed to act over a circular area of diameter d, and the allowable bearing pressure on the foundation be denoted by q, the required thickness of surfacing is given by the frequently-cited formula :

$$t = 0{\cdot}564\sqrt{\frac{W}{q}} - \frac{d}{2} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (3)$$

This formula takes no direct account of the shear strength of the surfacing or of the factors that govern the stability of the soil. Housel [12] has attempted to allow for these factors by assuming, firstly, that the load transmitted to the foundation is reduced by an amount corresponding to the shear strength of the cylinder ABCD of the surfacing (*Fig. 6*). The bearing capacity of the soil, which is assumed to be purely cohesive, is taken as $4c$, as shown in Appendix I. The safe bearing pressure that can be imposed on the surfacing can then be shown to be

$$p = 4c + 4mt/d \quad . \; . \; . \; . \; . \; . \; . \; . \quad (4)$$

where m denotes the shear strength of the surfacing.

If, however, when the foundation fails it tends to heave up against the underside of the surfacing over the area HDCG, the strength of the crust in shear will increase the safe bearing capacity of the surfacing. Assuming that AB = AE = d, it can readily be shown that the additional bearing capacity is $2mt/d$. In this case, therefore, the safe bearing pressure will be given by

$$p = 4c + 6mt/d \quad . \; . \; . \; . \; . \; . \; . \; . \quad (5)$$

The formulas deduced in Appendix I permit this result to be extended to more general soil conditions, the allowable bearing pressure on the surfacing being equal to the safe bearing pressure on the soil plus an amount varying between $4mt/d$ and $6mt/d$. The order of the ultimate shear strength of bituminous materials varies from 5 lb. per square inch upwards, whilst the shear strength of cement-stabilized soil may range from, say, 25 lb. to 100 lb. per square inch. According to Housel's formula, therefore, a surfacing only 4 inches thick could resist the punching shear exerted by a considerable wheel load. However, this type of failure will be rare, and recent work by Hubbard and Field [13] indicates that the failure of bituminous surfacings depends more upon the deflexion produced in the surfacing by the applied load than upon lack of stability in the soil. They state that this critical deflexion is between 0·5 inch and

0·6 inch, irrespective of the thickness of surfacing or of the diameter of the loaded area. For this reason, perhaps, equation (3) appears to have been more widely used than Housel's formula. Obviously considerable scope exists for further research into the design of flexible surfacings.

Aerodrome Drainage.

The drainage system of an aerodrome [14] has three main functions: the removal of surface water from the runways, the paved areas, and the site generally; the removal of subsoil water from the site; and the interception and diversion of subsoil and surface water originating in land adjacent to the site.

A single arterial drainage system is usually employed to deal with both storm-water and subsoil water. This arrangement is practicable because rainfall percolating through the soil does not usually reach the drains until after the storm-water has been carried away. Some American tests [15] at an aerodrome 170 acres in extent showed that on this site 40 minutes was a representative time of concentration of storm-water, whereas maximum discharge from a subsoil drainage system after heavy rainfall normally occurs after at least 4 to 6 hours. In the case of paved runways, channels or open-jointed pipes in trenches filled with gravel or rubble are generally laid at the sides of the runways and are connected at intervals, frequently through catchpits, to the arterial drains. Subsoil drainage proper usually has to be designed to conform to this general layout.

There are several reasons why it is desirable that the water-table should be maintained well below the surface. Firstly, the soil near the water-table tends to become saturated and soft; the water-table should therefore be kept away from zones where the soil is likely to be highly stressed: secondly, the deeper the water-table, the greater the storage capacity of the soil near the surface and the less easily will the soil become waterlogged under heavy rain; finally, frost causes cohesive soils to draw up water by capillary action, with the result that the soil frequently softens considerably during the thaw; the quantity of water that can be drawn up in this way is reduced when the water-table is lowered.

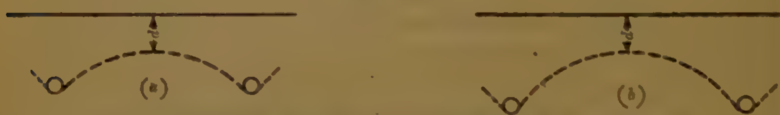
In designing subsoil drains, it is important to remember that such drains collect water only when they are at or below the water-table level. Subsoil drains in a homogeneous soil placed above the water-table cannot function, since the moisture in the capillary fringe is in a state of tension, as in a capillary tube, and can be removed only by evaporation or by transfer to lower levels through the capillary channels in the soil. The necessity for subsoil drains thus depends upon the position of the water-table under winter conditions relative to the finished surface.

Whilst surface water flow depends primarily upon the intensity and duration of rainfall, flow from the subsoil will also vary according to the nature and condition of the soil. Under normal summer conditions, except

on very pervious soils, surface evaporation, plant requirements, and the percentage absorbed by the top layer of soil, are considered to be capable of absorbing the first 1–2 inches of rainfall in any wet period following more than 7 days' drought. In winter, however, when surface evaporation is negligible and there is no plant growth, practically the whole of the incident rainfall must find its way to the drainage system unless there is some form of natural drainage. Subsoil drainage requirements of unpaved aerodromes thus depend chiefly upon winter demands. The actual flow to be dealt with depends upon the amount of rainfall, the storage capacity of the soil, and its permeability. American data suggest $\frac{5}{16}$ inch to $\frac{3}{8}$ inch per 24 hours, and calculations by Grubb [16] on a catchment in South-West England confirm a result of this order. The time of concentration was found in this case to be about 3 days.

Between drains, the water-table in a homogeneous soil rises as indicated in *Figs. 8* (*a*) and (*b*), and the depth and spacing of the drains for a given

Figs. 8.



RELATION BETWEEN SPACING AND DEPTH OF SUBSOIL DRAINS.

maximum height of water-table should depend upon the permeability of the soil, which in turn depends upon the fineness of the soil particles as indicated by mechanical analysis. The spacings suggested by the U.S. Corps of Engineers [7, 15], reproduced in Table I, are related to the mechanical analysis of the soil, and serve as a rough guide to the way in which the depth and spacing should be varied with the nature of the soil.

Frequently, however, the soil is not uniform, and the soil survey may reveal pervious beds overlying impervious beds or vice versa, or may indicate the presence of irregularly stratified layers of pervious and impervious materials. In the first case, subsoil drains should be spaced just above the impervious strata unless this lies below the depth at which the pipes would normally be placed if the impervious layer did not exist. Trenches without pipes cut through the impervious layer and filled with porous material would be sufficient in the second case. In a case of this type a silty clay overlying gravel was drained in the worst places by boring through to the pervious layer and filling the holes with gravel. The third type would require drains tapping the water pockets.

Soil surveys also enable water-table contours to be determined which give valuable information on the natural drainage of the site. *Fig. 7* illustrates a fairly flat aerodrome site fronting the sea, where the soil was a wind-blown sand overlying a rocky platform. The continuous rise in the water-table contours away from the sea indicates the existence of a

flow of subsoil water from the hinterland which could be intercepted by a cut-off drain behind the site. At the time of the survey (August), the ground water was 2 feet or more from the surface, but there was evidence from the nature of the plants that the central portion was waterlogged for a considerable period of the year. The regular nature of the contours is a characteristic feature of the sites so far examined, most of which have been on sandy soils.

Detailed information of surface contours may also indicate means of dealing with surface water before it has had a chance to soak into the subsoil. In one case, on a slightly sloping site with a silty clay subsoil, this was done by laying 6-inch by 9-inch box drains filled with gravel, 25 feet apart, centre to centre, nearly parallel to the contours.

TABLE I.

RECOMMENDED DEPTHS AND SPACING OF SUBSOIL DRAINS FOR VARIOUS SOIL CLASSES (U.S. CORPS OF ENGINEERS)[7, 15].

Soil class.	Mechanical analysis of soil*: per cent.			Distance between subsoil drains for various depths to invert of drain: feet.	
	Sand.	Silt.	Clay.	2–3 feet.	3–4 feet.
Sand	80–100	0–20	0–20	100–150	150–300
Sandy loam . . .	50–80	0–50	0–20	85–100	100–150
Loam	30–50	30–50	0–20	75–85	85–100
Silty loam	0–50	50–100	0–20	65–75	75–85
Sandy clay loam . .	50–80	0–30	20–30	55–65	65–75
Clay loam	20–50	20–50	20–30	45–55	55–65
Silty clay loam . .	0–30	50–80	20–30	40–45	45–55
Sandy clay	50–70	0–20	30–50	35–40	40–45
Silty clay	0–20	50–70	30–50	30–35	35–40
Clay	0–50	0–50	30–100	25–30	30–35

* U.S. Public Roads Administration Classification (see reference 2, p. 57).

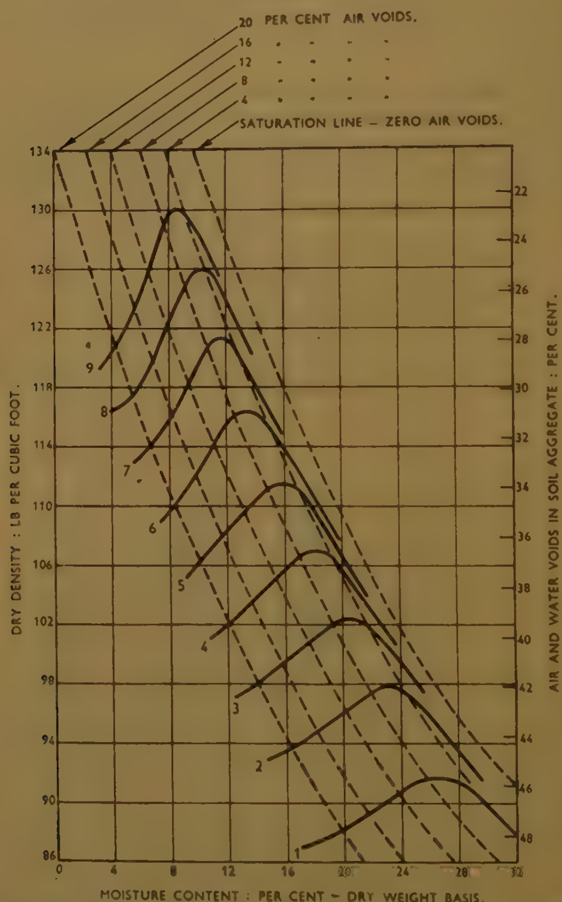
COMPACTION AND ITS RELATION TO EMBANKMENT CONSTRUCTION.

In constructing an embankment, the three main variables within the control of the engineer are the type of soil used, its moisture-content, and the methods employed for spreading and compacting. Choice of type of soil is limited, but construction methods and the moisture-content of the soil can be varied within wide limits if it is thought profitable to do so.

The essential qualities of an embankment—stability and freedom from excessive settlement—have still to be definitely related to the many factors involved, but the degree of compaction of the soil, that is, the closeness of packing of the soil particles, is probably one of the main factors governing the shear strength and the liability to settlement of the embankment. The degree of compaction can be readily determined in both the laboratory and

the field, and is measured by the weight of mineral particles in unit volume, that is, by the bulk density of the soil after deducting the weight of moisture present. This is commonly termed the " dry density " of the soil.

Fig. 9.



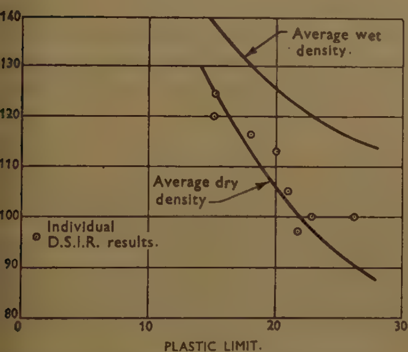
Average Relation between Dry Density and Moisture Content for Soils with Maximum Proctor Densities Differing by 5 lb. per Cubic Foot.

Proctor [17] was the first to show that the dry density of soil compacted under standard conditions is a function of its moisture-content. In the Proctor test the soil is rammed into a standard cylinder of 1/30-cubic foot capacity in three equal layers, using twenty-five blows of a standard rammer on each layer. Typical average curves for a wide range of cohesive soils are shown in *Fig. 9.* These curves were obtained by K. B. Woods [18] by

classifying the results of tests on numerous soils into groups according to maximum dry density, and averaging. For each group of soils, the dry density increases to a maximum at a moisture-content designated the "optimum moisture-content."

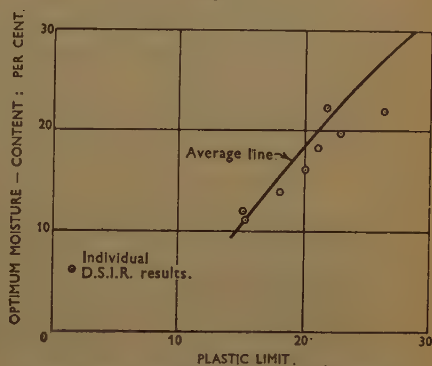
In *Fig. 9* curves of constant air-voids for a soil of particle density 2·70 have been superposed on Woods's curves. On the right-hand side of the diagram, the density-curves approach the line of zero air-voids, showing that mechanical compaction expels only air from cohesive soils. The difficulty of expelling this air increases as the percentage of air-voids becomes smaller, and also as the soil becomes stiffer with decrease in moisture-content. The compaction-curve therefore turns over as the moisture-content decreases, and the air-voids increase rapidly.

Fig. 10.



RELATION BETWEEN AVERAGE MAXIMUM COMPACTION DENSITY AND PLASTIC LIMIT OF SOIL.

Fig. 11.



RELATION BETWEEN OPTIMUM MOISTURE-CONTENT (DRY WEIGHT) AND PLASTIC LIMIT OF SOIL.

The much lower densities to which fine-grained soils are compacted under standard conditions are due to the fact that such soils become stiff at much higher moisture-contents than do the coarser-grained soils. Increased compaction will give increased density and has the same effect as the substitution of a more easily compacted soil [19].

Figs. 10 and *11*, plotted from data given by Woods, show how, on the average, the maximum dry density and the optimum moisture-content are related to the plastic limit of the soil. A few British results that have also been plotted lie closely about the American average curves. The index tests previously mentioned therefore yield a good deal of information as to the suitability of soil for filling. In Table II, a suggested classification of the quality of filling given by Woods has been related to the average plastic limit of the soil as determined from *Fig. 10*, and to the liquid limit determined from the average relation between liquid and plastic limits of British soils [3].

TABLE II.

RELATION BETWEEN COMPACTION AND INDEX PROPERTIES OF SOIL.

Quality of soil as filling.	Range of "Proctor" dry weight maximum densities: lb. per cubic foot.	Liquid limit: per cent.	Plastic limit: per cent.
Unsatisfactory or very poor	70–100	>65	>22
Poor	100–110	50–65	19–22
Fair	110–120	32–50	16–19
Good	120–130	24–32	14–16
Excellent	>130	<24	<14

A specification adopted by the State of Ohio [18] governing embankment construction is believed to be among the first to specify the degree of compaction required in relation to laboratory compaction tests and to the type of construction. The requirements of this specification are summarized in Table III. The requirements are less stringent for low embankments and for the coarser-grained types of soil. Measurements of the degree of compaction can readily be made in the field, and provide a useful check on construction procedure.

TABLE III.

MINIMUM SOIL COMPACTION REQUIREMENTS.

Maximum dry density of laboratory compacted soil*: lb. per cubic foot.	Minimum field compaction requirements (per cent. of maximum laboratory compaction).	
	Fills 10 feet or less in height and not subject to extensive floods.	Fills exceeding 10 feet in height or subject to long periods of floods.
<90	Not admissible for fill construction.	Not admissible for fill construction.
90–95	95	"
95–110	95	100
110–120	90	95
>120	90	90

* Standard compaction in Proctor cylinder.

Proctor's work has also been applied on a much more ambitious scale by controlling the moisture-content of the soil at or near the optimum value so that effort applied in compacting the soil is utilized to the best advantage. Under American conditions, water has usually to be added to obtain the optimum moisture-content of the soil. Sprinkling on the site has been found to be useful only within limits and irrigation of the borrow material is used whenever possible. In Great Britain, however, the

natural moisture-content of soil usually exceeds the moisture-content for optimum compaction (*Fig. 11*), and the control of moisture-content by the admixture of water is not generally possible. When the soil is too dry greater compaction can be obtained with heavier rollers, and it is significant that during recent years a considerable increase has occurred in the pressures used on the feet of sheepsfoot rollers : pressures of 675 lb. per square inch have been reported [18].

Conclusion.

In soil mechanics research considerable importance is attached to soil surveys, which play an essential part in enabling constructional experience to be correlated with laboratory investigations. Soil surveys should not, however, be regarded as a purely research activity, but should form an integral part of the preliminary investigations on all important engineering sites, and the ability to undertake this work should be regarded as an essential part of the training of a civil engineer. It scarcely seems necessary to stress the importance of systematic exploration of site conditions in view of recent events which have led to the abandonment of sites because the soil conditions were unsuitable. Such conditions could hardly have failed to be observed had a soil survey been made.

The practical application of soil mechanics involves, in certain cases, systematic tests on the site, and the provision of field laboratories for some of the more important civil engineering works seems likely to become common. For example, the time appears ripe for the adoption of some measure of control of the compaction of embankments on important roads. Field laboratories have already proved of value on stabilized-soil construction contracts where the supervising engineer must exercise close technical control of the process. During 1941, the Road Research Laboratory has had field laboratories working on six sites. Obviously the value of such field laboratories is not confined to soil investigations, but extends also to concrete and other construction materials where a closer measure of technical control is desirable.

The application of soil mechanics to aerodrome and road construction is only beginning to receive that intensive study which has characterized other branches of the subject. This is particularly true of aerodromes, where very little has yet been done to relate theory and practice, and where there is much need for further research. Even with our present knowledge, however, much information of practical value is available.

Acknowledgements.

The work described was carried out at the Road Research Laboratory of the Department of Scientific and Industrial Research, as part of the programme of research. Mr. H. Ll. D. Pugh, B.Sc., collaborated with the

Author in the mathematical work described in the Appendixes, and was responsible for the detailed calculations.

The Paper is accompanied by two photographs and nine sheets of diagrams, from which the Figures in the text and the half-tone page plate have been prepared.

APPENDIX I.

Expression of the Ultimate Bearing Capacity of the Soil in Terms of Its Mechanical Properties.

By H. Ll. D. Pugh, B.Sc., and A. H. D. Markwick, M.Sc., Assoc. M. Inst. C.E.

Bearing Capacity of a Loaded Strip on a Purely Cohesive Soil.—A simple approximate formula can be derived for the two-dimensional problem of a long foundation on a purely cohesive soil. Although it is not new, it is included in this Paper to illustrate how the bearing capacity of a soil arises.

Consider the element A (*Figs. 12* (*a*)), subject to a vertical pressure, p_1. If deformation is not to occur, the surrounding soil must exert a horizontal pressure, p_2, on the element. The minimum value of p_2 necessary to prevent deformation is governed by the consideration that in no direction can the shearing stress exceed the cohesive strength, c, of the soil. Resolving on any plane at angle θ to the vertical (*Figs. 12* (*b*)), the shear stress can be shown to be $(p_1 - p_2) \sin \theta \cos \theta$, the maximum value of which $(p_1 - p_2)/2$, occurs when $\theta = 45$ degrees.

Rupture will not occur on this plane if

$$(p_1 - p_2)/2 \leqslant c$$

or $$p_1 \leqslant p_2 + 2c \qquad (1)$$

Similarly, for element B, (*Figs. 12* (*a*)).

$$p_2 < p_3 + 2c$$

thus $$p_1 < p_3 + 4c \qquad (2)$$

If p_1 is assumed to be equal to the bearing pressure on the foundation, q_1, plus the pressure due to the weight of soil, wh, above the element, and p_3 to be equal to the pressure due to the weight of the soil, wh, and the superimposed load, q_2,

$$q_1 + wh \leqslant wh + q_2 + 4c,$$

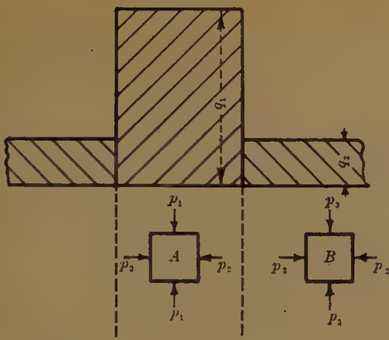
whence $$q_1 < 4c + q_2 \qquad (3)$$

This result agrees with the value deduced by Hogentogler and Terzaghi [9] by another method; the value found by Prandtl [20] and Hencky [21] by more rigorous analysis is $5{\cdot}14c$.

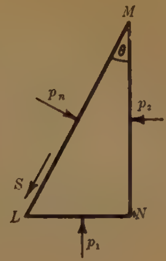
General Case of the Bearing Capacity of a Soil under a Loaded Circular Area.—The general three-dimensional case of a uniformly-loaded circular area bearing on a soil in which both frictional and cohesive resistance is present will now be considered.

First consider the element of soil in *Figs. 12* (*b*), which is subject to a stress p_1 acting normally to LN. A stress p_2 acting normally to MN is just sufficient to prevent deformation and can be calculated on the hypothesis that the shear stress on any plane must not exceed $p_n \tan \phi + c$, where p_n denotes the normal stress on the section, ϕ the angle of friction, and c the cohesive strength of the soil. This problem

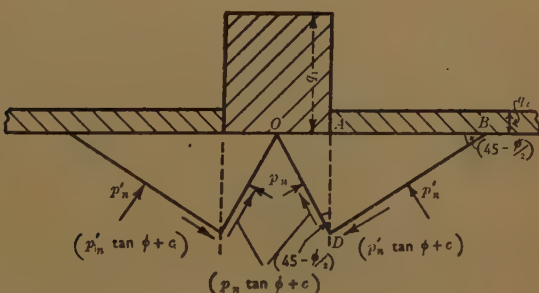
Figs. 12.



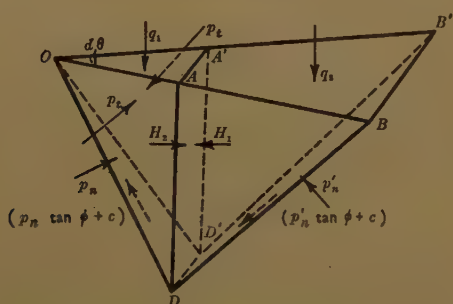
(*a*) STABILITY OF A LOADED STRIP OF SOIL.



(*b*) STABILITY OF ELEMENT OF SOIL.



(*c*) STABILITY OF LOADED CIRCULAR AREA SHOWING SECTION OF WEDGE OF RUPTURE.



(*d*) STRESSES ON ELEMENT OF WEDGE OF RUPTURE

was first solved by Mr. A. L. Bell, M. Inst. C.E.[22], and it can easily be shown, by resolving parallel and at right angles to LM, that failure will occur on the plane where $\theta = (\pi/4 - \phi/2)$, and that the relation between p_1 and p_2 on this plane is given by the equation :—

$$p_2 = p_1 \cdot \frac{(1 - \sin\phi)}{(1 + \sin\phi)} - \frac{2c \cos\phi}{(1 + \sin\phi)} \quad \ldots \ldots \quad (4)$$

The reasoning previously used cannot be readily applied to the three-dimensional case, since the value of the horizontal stress must diminish with increase in the distance from the centre of the load. The method used by Hogentogler and Terzaghi[9], has therefore been extended to the three-dimensional problem. The stability of the load is assumed to depend upon the equilibrium of a ring-shaped section (*Figs. 12* (*c*)), in which the angles of the wedges *ODA* and *ABD* are both made equal to $(\pi/4 - \phi/2)$, because at these angles the greatest risk of failure arises.

The stability of an element of the wedge of rupture (*Figs. 12* (*d*)) can now be considered. Consider first the prism *OAD D'A'* immediately under the load in the case of limiting equilibrium. The forces acting are the vertical stress, q_1, the normal and shear stresses on plane *ODD'*, the circumferential stress, p_t, and the horizontal stress, H_1. Resolving vertically and horizontally:

$$q_1 = \frac{p_n}{(1 - \sin\phi)} + \frac{c\cos\phi}{(1 - \sin\phi)} - \frac{2a\,\rho\cos\phi}{3(1 - \sin\phi)} \quad . \quad . \quad . \quad . \quad (5)$$

and

$$H_1 = \frac{p_n}{2(1 + \sin\phi)} - \frac{c}{2} \cdot \frac{\cos\phi}{(1 + \sin\phi)} + \frac{p_t}{2} \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Eliminating p_n from equations (5) and (6),

$$H_1 = \frac{q_1(1 - \sin\phi)}{2(1 + \sin\phi)} - \frac{c\cos\phi}{(1 + \sin\phi)} + \frac{a\rho\cos\phi}{3(1 + \sin\phi)} + \frac{p_t}{2} \quad . \quad . \quad (7)$$

Similarly the limiting equilibrium of sector *ABD D'B'A'* may be considered. The forces are similar to those acting on the element *OAD D'A'*, but in this case no circumferential forces are assumed, since the radial cracking of the ring of soil when it is displaced makes it impossible to rely upon this source of resistance to deformation. By resolving vertically and horizontally as before, and then eliminating p_n', it can be shown that:

$$H_2 = \frac{q_2(3 - \sin\phi)(1 + \sin\phi)}{2(1 - \sin\phi)^2} + \frac{c(3 - \sin\phi)\cos\phi}{(1 - \sin\phi)^2} + \frac{a\rho(1 + \sin\phi)^2(2 - \sin\phi)}{3\,(1 - \sin\phi)^2\cos\phi} \quad . \quad . \quad . \quad . \quad (8)$$

If q_1 represents the ultimate bearing capacity, $H_1 = H_2$, whence, from equations (7) and (8),

$$\frac{q_1(1 - \sin\phi)}{2(1 + \sin\phi)} \quad \frac{q_2(3 - \sin\phi)(1 + \sin\phi)}{2(1 - \sin\phi)^2} + \frac{4c}{(1 - \sin\phi)\cos\phi} + \frac{a\rho(1 + 6\sin\phi - 3\sin^2\phi)}{3(1 - \sin\phi)^2\cos\phi} - \frac{p_t}{2} \quad . \quad . \quad . \quad . \quad (9)$$

Consideration of the numerical coefficients of the first and last terms shows that the value attached to p_t has an important effect upon the bearing capacity. Let p_1, in equation (4), represent the vertical stress at a depth x from the surface, and let p_3 in this equation represent the horizontal stress p_{tx} at a depth x. Equation (4) then becomes:

$$p_{tx} = (q_1 + \rho x)\frac{(1 - \sin\phi)}{(1 + \sin\phi)} - \frac{2c\cos\phi}{(1 + \sin\phi)} \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Now p_t is the average value of p_{tx} taken over *OAD*, and on substituting in equation (4) it can be shown that

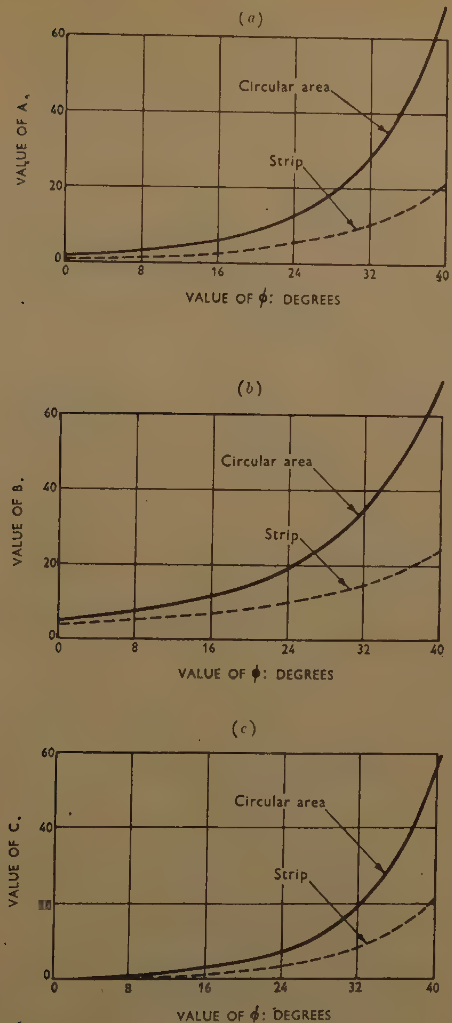
$$q_1 = \frac{q_2(3 - \sin\phi)(1 + \sin\phi)^2}{2(1 - \sin\phi)^3} + \frac{c\cos\phi}{(1 - \sin\phi)}\left[1 + \frac{4}{(1 - \sin\phi)^2}\right] + \frac{a\rho\cos\phi}{3(1 - \sin\phi)}\left[\frac{(2 - \sin\phi)(1 + \sin\phi)^2}{(1 - \sin\phi)^3} - \frac{3}{2}\right] \quad . \quad . \quad . \quad . \quad (11)$$

Equation (11) is of the general form:

$$q_1 = Aq + Bc + Ca\rho \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

where A, B and C are numerical coefficients depending on ϕ. The values of these coefficients are plotted against ϕ in *Figs. 13* (*a*), (*b*) and (*c*). In all cases the values of

Figs. 13.



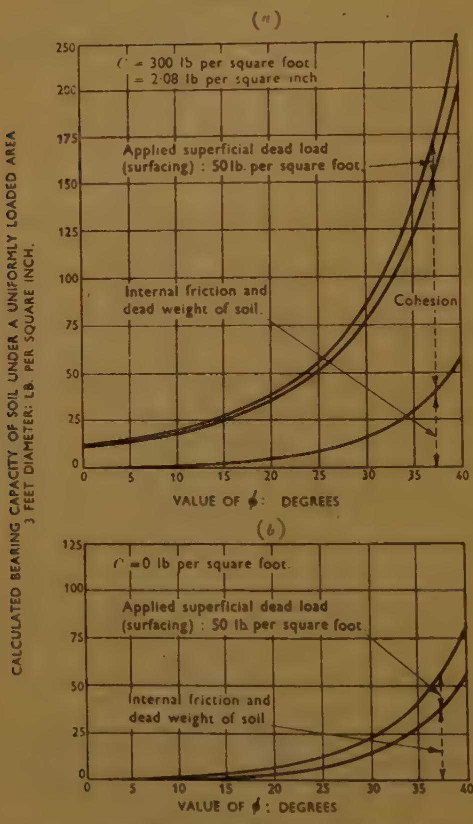
VALUES OF COEFFICIENTS IN EQUATION (12)
FOR LOADED CIRCULAR AREA AND LOADED STRIP.

the coefficients increase rapidly with increase in the angle of friction. The resistance due to cohesion is also seen to be greatly increased when internal friction is present. For example, in Table IV, which gives the calculated bearing capacities for a number of typical soils, comparing a soft clay with a sand containing a clay binder, both of which are assumed to have the same cohesive strength (2·78 lb. per square inch), the bearing capacity of the clay ($\phi = 4$ degrees) is only 17 lb. per square inch, whereas the bearing capacity of the sand ($\phi = 30$ degrees) is 87 lb. per square inch.

Figs. 14 illustrate the importance of the cohesive strength of the soil, and by inference the prime importance of drainage. The effect of a confining pressure such

as that due to a flexible surfacing is important (Table IV) only in the case of sand, where the bearing capacity is thereby increased from two- to nearly four-fold. When the soil possesses cohesive strength this factor is of little importance. The effect of the density term, $C a\rho$ is likewise important (*Figs. 14*) only in soils having a low cohesive strength. It is therefore in such soils that the reduction of density due to buoyancy when the water-table rises must be taken into consideration.

Figs. 14.



CALCULATED ULTIMATE BEARING CAPACITY OF SOILS OF VARIOUS PROPERTIES.

In the special case of a purely cohesive soil when $\phi = 0$ and when $q_2 = 0$,

$$q_1 = 5c + a\rho/6 \qquad (13)$$

A more rigorous analysis by Hencky [21] of this case, when the effect of the weight of the material is neglected, leads to the value 5·64c for the bearing capacity of the soil as against 5·14c for the two-dimensional case. The similarity between the numerical values of the coefficients of c for the two- and three-dimensional cases suggests that the formula for the bearing capacity of an elliptical bearing surface of a purely cohesive soil would differ little from either.

In addition to the values of A, B, and C, computed from equation (11), values are shown by the dotted curves in *Figs. 13* for the corresponding equations originally deduced by Bell [22] and by Hogentogler and Terzaghi [9] for the two-dimensional case. When friction is present the ultimate bearing capacity of the loaded circular area is

TABLE IV.

CALCULATED ULTIMATE BEARING CAPACITIES OF TYPICAL SOILS UNDER LOADED CIRCULAR AREA.

Soil.	Typical mechanical properties of soil. Cohesion c. lb. per square foot.	Cohesion c. lb. per square inch.	Angle of internal friction: degrees.	Calculated bearing capacity: lb. per square inch. Bare ground ($q_2 = 0$). $a = 6$ inches.	Bare ground $a = 18$ inches.	With surfacing ($q_2 = 100$ lb. per square foot). $a = 6$ inches.	With surfacing $a = 18$ inches.	Increase in bearing capacity due to surfacing: per cent. $a = 6$ inches.	Increase $a = 18$ inches.
Clay, almost liquid . . .	100	0·69	0	3·5	3·6	4·6	4·7	31	30
Clay, very soft .	200	1·39	2	7·7	7·9	8·9	9·1	16	15
Clay, soft . .	400	2·78	4	17	17·1	18	19	9	9
Clay, fairly stiff.	1,000	6·94	6	46	47	48	49	4	4
Clay, stiff . .	1,500	10·42	8	77	78	79	80	3	3
Clay, very stiff .	2,000	13·89	12	127	128	130	131	2	2
Silts, wet . .	0	0·00	10	0·4	1·2	2·8	3·6	700	300
Sands, dry . .	0	0·00	34	8·6	26	33	50	378	193
Sand predominating with some clay. .	400	2·78	30	87	97	102	113	18	16
Sand–gravel mixtures cemented	1,000	6·94	34	290	308	314	332	8	8

This table is based on a soil of density 100 lb. per cubic foot.
a denotes equivalent radius of the area of contact of the tire.

about twice that of the loaded strip. This agrees with an approximate analysis made by Terzaghi [23] on the lines of *Figs. 12* (*a*) when for a purely granular soil a value of twice that found for the strip load was obtained. These results suggest that the bearing capacity of a circular area will be greater than that of an elliptical area of the same width.

APPENDIX II.

APPLICATION OF DIMENSIONAL ANALYSIS TO THE BEARING CAPACITY OF SOIL UNDER AEROPLANE TIRES.

The general relationship between the variables governing the bearing capacity of a homogeneous soil under an aeroplane tire may be investigated by dimensional analysis. This method involves the minimum of assumptions as to the mechanism of failure and requires only a knowledge of all the factors entering into the problem, which are assumed to be: P, the wheel-load which can just be supported by the soil; v, the velocity of the wheel; d, the major axis of the tire ellipse; w, the density of the soil; c, the cohesive strength of the soil; ϕ, the angle of internal friction in the soil; and g, the gravitational constant.

If discussion be restricted to tires whose contact ellipses are geometrically similar, the wheel-load is related to tire-pressure by the relation $P \propto pd^2$. Thus the tire-pressure,

p, is a dependent variable and can be omitted from the analysis. The theory of dimensional analysis permits the general relation between the seven variables mentioned above to be expressed by a functional relationship between four (7–3) dimensionless groups chosen from these variables: this relationship may be expressed as follows:—

$$\frac{P}{wd^3g} = \psi\left(\frac{v^2}{dg}, \frac{c}{wdg}, \phi\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where ψ is an unknown function that must be determined independently either by more detailed analysis or by experiment. For any given soil, w, c, ϕ, and g are constant. For model experiments to be possible, all the non-dimensional groups must be maintained constant. Since the term c/wdg is present, it follows that in a given soil d must be constant: hence model experiments are not possible in the general case if the same soil is used.

In the special case when $c = 0$, and when the tire is stationary,

$$\frac{P}{wd^3g} = \psi_1(\phi) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The conditions for similarity are that P/wd^3g and ϕ should be constant. For a given soil both ϕ and w are constant, so that the other condition for similarity is that $P \propto d^3$. It follows from the relation $P \propto pd^2$ that this condition may be expressed $p \propto d \propto P^{\frac{1}{3}}$. Hence the inflation-pressure on a granular soil may be increased in the ratio of $P^{\frac{1}{3}}$ without affecting the stability of the soil.

For a purely cohesive soil it has been shown in Appendix I that, neglecting the weight of the soil, $p \propto c$, so that for a given soil the permissible inflation-pressure is independent of the wheel-load and depends only upon the cohesive strength of the soil.

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A report of the Discussion on the Papers by Messrs. Cooling and Markwick will be printed in the April Journal.—Sec. Inst. C.E.

Paper No. 5293.

"The Reconstruction of the Engineering Equipment of the Harrogate Royal Baths."

By Arthur Henry Barker, B.Sc., B.A., M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*)[1]

The Paper gives a detailed account of the recent reorganization by the Author of the complicated services at this well-known Spa building. The district around Harrogate is unique in the possession of a remarkable series of about 100 mineral springs of high curative value, all rising from the earth within an area of about 4 square miles, most of them within a few acres, and many within a few yards of one another. A plan showing these positions is given in the Paper.

The water from each separate spring has been found to be constant in quantity, temperature, and composition, which have not varied since the first recorded measurements were made one hundred years ago; but there is a wide variation in the chemical constituents of the waters, even between springs a few yards apart. Detailed analyses of five typical springs are given in the Paper.

The water is independent of rainfall, being generated in the interior

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[1] The full MS. and illustrations may be seen in the Institution Library.—Sec. Inst. C.E.

of the earth. It is formed during the decomposition by intense heat of granitic rock which, originally solidified by gradual cooling of the earth's molten core, was again disintegrated by some enormous internal convulsion many millions of years ago which projected portions of the previously solidified granite shell into the interior white-hot molten core at an inconceivable temperature. Water is known to be one product of the decomposition of granite by great heat, and water so formed is forced by interior pressure through cracks in the earth's crust up to the surface. Its chemical composition is due to contact with the variety of solid soluble salts in enormous masses met with on its way.

This continuous constancy of composition and temperature is the result of the absence of local seismic disturbances during the past hundred years, which has presumably maintained each spring in its own constant channel, with practically constant conditions all the way to the surface.

The curative value of these waters was discovered several hundred years ago, and their use by the medical profession has evolved gradually from individual exploitation to the present highly-organized condition.

The outflow of each spring is first collected in a small local reservoir and thence pumped through large lead pipes buried under the streets to one or other of the three large reservoirs in various convenient higher parts of the district, whence it flows to a central point in the lower part of the town where there is a fine installation of hundreds of baths, sprays, drinking-fountains, and other facilities. The pipes have been in existence for many years, and their interior surfaces, when exposed, have been found to be covered with a deposit of scale which appears to have protected the interior surface of the metal from continuous corrosion, enabling the pipes to be used for long periods without considerable repairs or renewals. They are at present so far serviceable that no serious disturbance of the street services was found necessary for their continued use after the recent reorganization.

The chemical constituents of the waters are numerous, consisting for the most part of sulphur compounds, especially sulphuretted hydrogen, and various salts of sodium, potassium, magnesium, calcium, and barium. Some of the waters are rich in iron, and it is probable that a certain proportion of "heavy water" is also present.

The waters have highly evanescent properties and exposure to air reduces their strength in a material degree, largely owing to the escape of the sulphur compounds. These are of highly corrosive character and will rapidly attack and dissolve all ordinary metals such as steel, copper, lead, zinc, brass, etc.; and they also liberally deposit some of their constituents in the form of gritty scale or sludge. Their handling as an engineering proposition is therefore difficult; for instance, an ordinary valve or pump is very largely destroyed after six months' use and has to be re-seated or rebuilt.

The pipes composing the highly complicated original system were of

ordinary metals, especially copper and lead, and were rapidly eaten away in parts and rendered leaky so that they could not be used continuously without constant repairs, which, owing to the great congestion of the numerous necessary pipes, were always difficult and very expensive. The reconstruction of this vast mass of pipework was designed with the objects of extending the system to large new additions recently completed, and at the same time reducing the enormous cost of continual repairs rendered necessary by the corrosive character of the liquid.

The types of water from the springs are divided into three main classes—

(1) So-called "strong sulphur."
(2) "Alkaline sulphur."
(3) "Chalybeate" waters.

The products of the various springs in the district, the composition of each of which is accurately known, are divided into these three categories and are collected into separate reservoirs for each type.

The normal requirements of a Spa building, including heating, ventilation of three different kinds, gas, electric power, drainage, ordinary town's water, steam, and condensed water, combined with a supply of six different kinds of water and in some cases chemical gases, carbon dioxide, compressed air, and high-pressure water (at 30–50 lb. per square inch) as required by the multifarious methods of treatment specified and supervised by a large and highly-specialized team of medical experts, obviously produce an immense congestion of tanks, pipes, ventilating trunks, valves, etc., since every service has to be supplied to each of the large number of treatment-rooms. All pipes and fittings must be entirely concealed from the patients and installed so that any necessary repairs can be effected without disturbing the structure of the building.

General Description of Treatments.—The treatments comprise, among others :—

1. Plain baths of different compositions, at controlled temperatures, in which the patient is immersed for specified times.

2. Similar baths with special conditions such as :—
   (*a*) projection of a continuous stream of water, at a different temperature, over special parts of the patient's body while maintaining a constant temperature in the bath ;
   (*b*) the injection of extraneous gases such as carbon dioxide over parts of the body of the patient under the water ;
   (*c*) the production of foam, in which the patient is immersed ;
   (*d*) the provision of mud baths consisting of medicated and pulverized peat intimately mixed with water.

3. (*a*) The provision of pads of various medicated mixtures applied to various parts of the patient's body, such as stomach or liver ;
   (*b*) the projection of highly subdivided sprays of liquid maintained

at high pressure and constant temperature for a prolonged period over the whole or part of the patient's body, accompanied by vigorous massage;

(c) the projection of powerful high-velocity jets of water over the patient's body.

4. Internal treatments in which liquids of exact composition are injected into the patient's bowels.

5. A large number of ordinary treatments, such as Turkish, Russian, steam, electric, and light baths.

The first problems presented by the reconstruction were (i) to find suitable materials for the pipes, valves, and fittings, which should be non-corrodible and easy to connect with other pipes by some kind of union, also non-corrosive and inexpensive to fit and repair; (ii) to provide means for keeping the water as much out of contact with air as possible.

Other major problems were the design of pumps and valves not involving the use of ordinary glands or other rubbing parts, which are found to be impossible to maintain in permanent repair owing to the abrasive character of the liquid dealt with and the gritty solids deposited from it.

Deterioration of Medicinal Waters.—As every avenue by which the sulphur elements can escape into the air entails a deterioration in the quality of the water, it is of the highest importance to prevent such exposure.

The most desirable material for the pipes and valves was determined by a prolonged series of analyses and practical trials of various experimental mixtures of metals, and the subjection of each to the action of the waters for measured periods, with careful weighing of samples before and after exposure to determine the respective degrees and speeds of corrosion. With this material it was found possible to increase the life of the pipes from about 12 months (as previously found in practice) to a calculated period of about 200 years—that is indefinitely. The thickness of the pipe made of the special material is 24-gauge, whilst the copper pipe which it replaces was 10-gauge; the special material, though more expensive per pound than copper, is cheaper per foot run. A means of jointing the pipes was also discovered, which enables any pipe to be taken apart when necessary and rejointed easily and rapidly. In addition, chemical means of cleaning the pipes without destroying them were devised.

Reservoirs.—In every case where it was necessary that the surface of the liquid should be free and offer as little exposure to the air as possible the reservoir was provided with a wooden raft formed of flat planks closely jointed together at their edges and covering the surface of the water in the tanks except for a small clearance at the edge. These rafts were heavily coated with a wax which would be unaffected by the liquid, and were made with a clearance of about $\frac{1}{8}$ inch all round, so that whilst they float

freely on the surface of the water they allow the latter to rise and fall freely, the only exposure to the air consisting of a narrow rim all round the rafts as small as is necessary for clearance.

Design of pumps.—A type of centrifugal pump was adopted, having bearings outside the liquid, which enabled the gland to be dispensed with, its place being taken by a clearing hole slightly larger than the spindle surrounded by a small collecting chamber and overflow pipe which conducts the escaping liquid back into the reservoir from which the liquid is being pumped, with very slight exposure to the air.

The pipes from the large reservoirs to the bath installation conduct the water through non-corrodible ball valves of special design which in order to avoid disturbance of the surface, discharge into the bottom of large concrete settling tanks reinforced with non-corrodible steel: thus the minimum of sediment is carried into the smaller service pipes in the building. From these settling tanks two kinds of water are raised separately, by the pumps described above, to two similar large storage-tanks in the new tower of the building, through similar non-corrodible ball valves. From these tanks the various qualities of water are conducted separately in large sub-ceilings between the floors of the buildings; the space available is about 12 feet wide by 3 feet high, and in it all the various pipes, valves, conduits, switches, drains, flues, etc., are conducted all over the buildings to the various baths.

The pipes are arranged so that any one of them can be reached by a mechanic, who usually has to operate from a prone position underneath the range; they are all suspended by swinging links from slotted transverse girders at the roof of the pipe track in such a way that each of them can be fixed in any position, and expand and contract freely, and so that any section of any pipe can be removed and replaced without difficulty. Samples of the water can be taken continuously to enable the degree of deterioration of the water at each point to be determined.

Jointing.—The jointing of the pipes is effected by a special type of joint in which the body of the pipe is formed accurately cylindrical and about 1/1000 inch smaller than the bore of the socket. A ring of solder is formed in a recess in the interior surface of the socket, the jointing being effected by applying a blowpipe to the exterior with the pipe in place and treated with flux; this melts the solder, which flows by capillary attraction all over the joint, forming a connexion which is stronger than the pipe itself.

Valves.—A special type of valve with no gland was adopted, closing being effected by a flexible diaphragm of thick and soft rubber forced by an exterior screw on to a weir extending across the bottom of the interior of the valve body. The depression of this diaphragm makes a perfect joint on the weir in spite of any accumulated deposit on the latter; it can be removed and replaced in a few minutes, and the degree of opening of the valve can be observed from the outside.

Control of water-temperature.—An important problem arose in connexion with the provision of means for spraying water at an absolutely constant temperature over the bodies of patients, avoiding the danger of variation of pressure in either the cold or the hot pipe serving the spray or jet, with a consequent variation in the temperature of the mixture. The hot water is maintained at a constant temperature of about 150° F., whilst the cold supply is taken from separate tanks or direct from the town mains. High-pressure jets are obtained by passing the water from the mixing valve through a small auxiliary boosting pump, operated by a variable-speed motor so that any desired pressure up to the maximum can be obtained, as indicated by a local pressure-gauge. Thermostatic control is effected by a specially-designed mixing valve which regulates automatically the mixture of cold and hot water. In the event of a drop in pressure or of failure of either supply, which might cause the temperature to change rapidly above or below the desired value, the whole of either supply is regulated or automatically shut off and is not restored until conditions are again normal.

The heating and ventilation of these large buildings is necessarily complicated by the fact that complete and rapid individual control of the temperature in each of the large number of rooms is required. At some periods only a few of the rooms are occupied, whilst at others they are filled to capacity. To leave full heat on continuously would involve considerable waste of fuel.

The ventilation is also complicated by the fact that the air from certain of the rooms must on no account be allowed to mix with the general air of the building. In some rooms highly objectionable smells are created and in others large volumes of vapour. Completely separate installations are therefore necessary, with no intercommunication through the flues between the various portions of the building. These conditions are fulfilled as follows :—

Heating is effected by a system of vacuum steam, that is, steam below the pressure of the atmosphere. The condensation return pipe is maintained under a higher degree of vacuum by means of an ejector type of centrifugal pump, which not only condenses the steam but also returns it to the boiler feed tank. The control is fully automatic and is effected by the simultaneous thermostatic regulation of steam-pressure accompanied by regulation of the difference in pressure between the steam and condensation pipe-lines. Thus the general temperature of the radiators is controlled. The Paper contains a full description of the method.

Fresh air is delivered in two trunks, one carrying filtered air warmed to a predetermined temperature and the other cold air. The air delivered to each room is a mixture in hand-regulated proportions of the two so that the mixture can be delivered at any temperature desired by the occupant. The control of the dampers is effected from each separate room by Bowden wire, enabling any room to be heated rapidly to any desired degree from a basic temperature which is constantly maintained by the radiators.

For financial reasons some portions of the equipment mentioned were omitted, for addition after the war.

The Paper is accompanied by a map and seven sheets of drawings.

INGENUITY COMPETITION, 1941.

"Pile Driving without a Pile Frame."

By Stanley Pearson, M. Inst. C.E.

The operation to be carried out consisted of the driving of 6-inch by 6-inch timber piles ranging, according to the ground conditions, from 18 feet to 26 feet in length, to form the main support for the new revetment of a badly eroded river-bank. The piles had to be driven accurately along a predetermined line, spaced at equal centres, be vertical all ways, and driven to a uniform set. The line began in the old river-bank, continued out into the stream, sometimes to a distance of 18 feet from the eroded bank, and finished again in the hard ground of the river-bank. In the worst cases the piles were driven through water and silt approximately 8 feet deep. The ground-level at the top of the river-bank averaged about 18 feet above normal water-level and the slope of the bank ranged from 2 : 1 to almost vertical. The width of the river ranged from 40 feet to 60 feet, the final width as designed being 42 feet.

Access to the site was bad, the immediate ground in the vicinity being plough-land with no roadway running along the top of the river-bank. A dragline with a 40-foot jib was in the vicinity, being used for muck-shifting to widen the narrower stretches of the river, prior to the construction of the new revetment work. In these circumstances the use of an ordinary pile frame with power-driven hammer and derrick and power plant would have proved extremely costly.

The erratic behaviour of the river added to the difficulties. From observations taken before the commencement of the work, the following rises had been noted :—

16 feet rise from normal water-level to maximum level in three hours ;
the river-level remained at a height of 6 feet above normal for 4 days in succession ;
the usual daily fluctuation during fair weather ranged from normal to 3 feet 6 inches above normal.

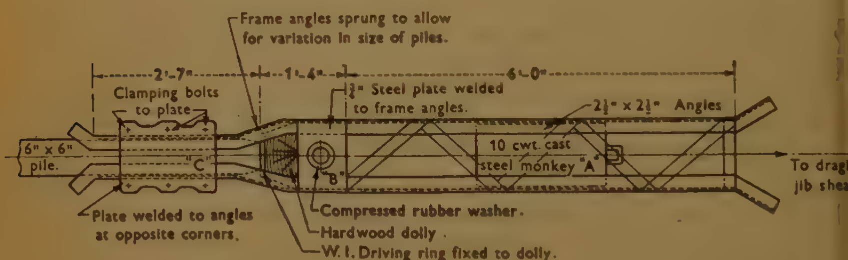
It was therefore decided to carry out as much work as possible from the

top of the river-bank at ground-level, and that no plant should be placed in the river-channel.

A guide-frame was constructed, consisting of four 2½-inch by 2½-inch angles, about 12 feet long, braced apart with steel stays welded on the outside of the angles to form a box, down the centre of which could slide a 10-cwt. cast-steel monkey, A, 1 foot by 1 foot by 2 feet 3 inches in length (*Fig. 1*). The internal dimensions of the box section were 1 foot ½ inch square.

Two solid steel plates, B, 1 foot 1 inch by 9 inches by ¾ inch were welded to the frame angles on the outside, with their tops 6 feet 6 inches from the top of the frame. From the bottom of these plates for a distance of 7 inches the angles were sprung together, reducing the internal dimensions of the section to 6½ inches by 6½ inches. From this point downwards

Fig. 1.

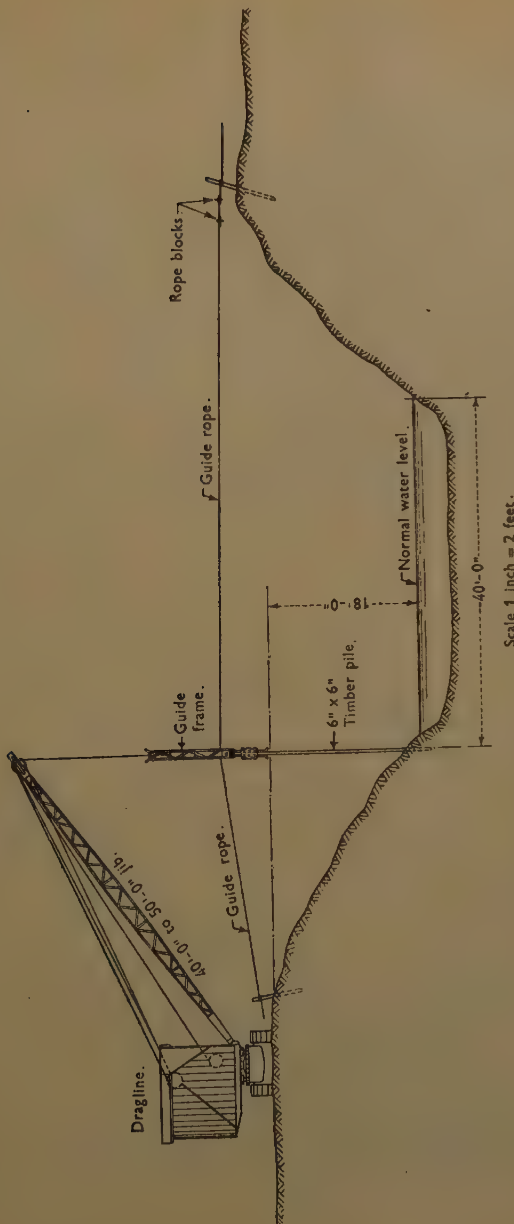


DETAILS OF GUIDE FRAME.

the angles were not braced in any way, being left free to exert lateral pressure on the pile when inserted in the end of the section.

In the length of the frame between the top of the plates and the lower end of the sprung portion of the angles, a hardwood dolly was fitted, ringed at the bottom with a wrought-iron driving-ring. This dolly was maintained in position by bolts running through it and through the centre of plate B, the shock of the blow being taken by hard compressed rubber bushes between the bolts and the plates. The remaining lower portion of the frame formed a clamp to maintain the pile in position. This was achieved by two plates of steel, C, 1 foot 3 inches wide by 1 foot 6 inches deep and ¾ inch thick, which were fixed on each side of the frame by welding to opposite angles only and drilled to take four bolts, fitted with nuts and washers, which passed through the plates outside the angles. The extreme ends of the angles were splayed out for a few inches at both the top and the bottom of the frame, to facilitate the entrance of the dolly at the top and the pile at the bottom.

Fig. 2.



LAYOUT OF PILE DRIVING. EQUIPMENT FOR USE WITH DRAGLINE.

Three ropes were then secured to the top of the section, of sufficient length to maintain it steady during driving, from two positions on the near bank of the river and from a third position on the far bank.

The operation of pitching a pile is as follows. The dragline is brought to the top of the bank immediately above the position in which the first pile is required to be driven, the bucket is removed from the winding-rope, and the drag rope is disconnected, since it is not required in the operation (*Fig. 2.*) The guide-frame is now laid horizontally on the ground, and the monkey is placed in the larger end ; the winding rope of the dragline is then attached to the monkey and a steel bar is placed through the stays of the guide-frame and above the monkey, so that the whole frame can be lifted by the dragline, the monkey being held in the frame. A timber pile is then inserted in the smaller end of the frame and rammed home, so that the top of the pile is in contact with the lower face of the dolly. The bolts in the lower plates, C (*Fig.1*), are tightened and the pile is made fast in the frame.

The dragline driver then pulls in his winding-rope and lifts the frame, monkey, and pile in one unit, and slues them to the position for driving. Three men are posted at three anchorages previously placed, two on the near bank of the river and one across the river. Rope-blocks are attached to the anchorages and by means of the three guide-ropes already fastened to the frame the pile is guided to and braced in the correct position. When the driver receives the signal that the pile is in the correct position he releases his brake, the pile-frame and monkey are dropped, and the pile is pitched. The bar holding the monkey in position is then pulled out of the frame, and the monkey is raised and released by the driver. *Figs. 3* and *4* illustrate the operation.

After the pile has been driven to the required set the bolts holding the pile are slackened off, the bar is inserted above the monkey in the frame, and the frame is pulled from the top of the pile, so that all is ready to repeat the operations for the next pile. After two piles have been driven, walings or guide-rails are fixed to the piles already driven to simplify the pitching of successive piles.

Thus work can be carried on independently of the small rises in the river, whilst there is no plant in the river channel to suffer damage, or to be lost by the higher rises, the only heavy plant required being the dragline working on mats, well above flood-level and completely mobile over the bad ground ; and in this case the machine was made to serve a dual purpose.

The following figures, which are based upon pre-war prices, do not indicate the comprehensive cost of the job, but they demonstrate the relative cheapness of the method in carrying out this particular job :—

Fig. 3.

Fig. 4.

	s.	d.
Cost of dragline for one day, including driver, fuel, etc.	72	9
Four labourers, per day	37	0
One ganger, per day	11	5
	121	2

Number of piles, average 20 feet long, driven in one day . . .	8
Total penetration, at 16 feet average per pile	128 feet.
Cost per foot of penetration : labour and plant only	11½*d.*

This equipment was made and used on a contract for works on the river Mersey by Messrs. A. Monk & Co., Ltd., of Padgate, Warrington, who collaborated in the design and construction of the plant.

LECTURE ON

"The Importance of Management Training for Engineering Students."

Delivered by

LYNDALL URWICK, O.B.E., M.C., M.A., F.I.I.A.

at the University of Cambridge on the 23rd January, 1942.

Being a Lecture in the series of Lectures on Engineering Economics, Management, and Aesthetics arranged by the Council of The Institution in conjunction with the Senate of Cambridge University.

IT has once or twice been my good fortune to be invited to lecture at one or other of our older Universities. But a life largely devoted to practical pursuits has left me with an attitude towards, and an experience of, Universities which are inevitably undergraduate. On the few occasions when I have addressed an Oxford student audience I have felt something of the helpless astonishment with which a baby might find itself trying to bath its mother. To-night my ordeal is even more shattering. With the friendly abusiveness which my own University employs to conceal its own defects, this seat of learning is sometimes described as that " curious technical college in East Anglia," a tacit confession that as a preparation

for practical life we recognize certain limitations in our own curriculum. When I add that the elementary mathematics of the School Certificate examination defeated me on no less than two separate occasions, you will appreciate the qualms with which I face a Cambridge audience largely composed of engineering students.

I have, however, found a certain comfort in two facts. In his Presidential Address to the Institution of Civil Engineers [1] Professor C. E. Inglis emphasized that mathematics is a peculiar subject in that "to most individuals it presents a ceiling above which it is impossible to rise." He also said, "Engineering is now shaping the destiny of civilization; it has vast potentialities for both good and evil and, side by side with his scientific training, a student should have his interest stimulated towards the humanitarian, the economic, and even the ethical, responsibilities of the profession he is about to enter."

Thinking over the ideas I had intended to develop, I appreciated that they were but a footnote to that general statement, which I will take as my text.

What do we mean by management? In 1915 one of the most remarkable men of our century died at his home outside Philadelphia. Frederick Winslow Taylor was an American engineer who, on the technical side alone, achieved more than would have filled most lifetimes. His invention of high-speed steel revolutionized machine-shop practice; his work on belting was equally original. But it is his work on management which has influenced every country in the world.

The nature of that work is still imperfectly understood. Men talk about the "Taylor system" as though Taylor had invented a new kind of card index, or some novel method of book-keeping—a rigid formula applicable to any situation and warranted to cure all industrial ills from rickets to rheumatism, or perhaps, in this context, I should say from suspicion to sabotage. It is true that there is scarcely a factory in the world which has not, wittingly or unwittingly, applied some of the devices which he developed. Wherever you find a planning department, time- or motion-study, instruction cards, standard costs, process analysis, and half a hundred other modern developments—there the mind of Taylor is made manifest in the practical routines of other men, the majority of whom have scarcely heard his name.

But the notion that these things constitute "a system," that they are Scientific Management—the somewhat unfortunate title given to his concepts—is merely a symptom of our common intellectual weakness. We no longer attribute a rainstorm to the thunder god, or stick pins into the waxen image of our pet aversion. We are still infantile enough to seek well-being in proprietary remedies if the advertising is sufficiently attractive, instead of trying to learn and striving to keep the rules of health.

[1] Journal Inst. C.E., vol. 17 (Session 1941–42), p. 1 (Nov. 1941).

It is far easier to earn a million pounds with pills for something or anything, than a tenth of that sum as an honest doctor. And politically, the party without an " -ism " to make nonsense of its policy is unlikely to secure the support of the electorate.

Taylor knew better. Towards the end of his life, when he was being hazed by a Congressional Committee, he exploded into a remarkable testimony as to his real scale of values :

> " Scientific management is not any efficiency device . . . nor is it any bunch or group of efficiency devices. It is not a new system of figuring costs ; it is not a new scheme of paying men ; it is not holding a stop-watch on a man and writing things down about him ; it is not time study ; it is not motion study nor an analysis of the movements of men ; . . . it is not any of the devices which the average man calls to mind when scientific management is spoken of. . . . In its essence, scientific management involves a complete mental revolution on the part of the working man engaged in any particular establishment or industry. And it involves an equally complete mental revolution on the part of those on the management's side—the foreman, the superintendent, the owner of the business, the board of directors. . . . Both sides must take their eyes off the division of the surplus as the all-important matter, and together turn their attention toward increasing the size of the surplus. . . . Both sides must recognize as essential the substitution of exact scientific investigation and knowledge for the old individual judgment or opinion, either of the workman or the boss, in all matters relating to the work done in the establishment. . . . Scientific management cannot be said to exist, then, in any establishment until after this change has taken place in the mental attitude of both the management and the men. . . ."

" A mental revolution "—that, and no less, was his real aim. But revolutions are not inventions. Whilst a single individual may detonate the charge, the charge must first be laid. In the introduction to his famous paper " A Piece-Rate System "—read to the American Society of Mechanical Engineers in 1895 [1]—Taylor mentioned six other men whose experiments on various aspects of business administration had been incorporated in his work. They covered such different subjects as progressing, a specialized employment department, a messenger system, a mnemonic system of order numbers, inspection, apprentice training, and shop returns. Prior to his Paper, two other engineers had addressed the Society on analogous questions, H. R. Towne in 1886 on " The Engineer as an Economist [2] " and F. A. Halsey in 1891 on " The Premium Plan for Paying Labour [3]." In Great Britain as early as 1887 an engineer and an

[1] Trans. Am. Soc. Mech. E., vol. xvi (1894–95), p. 856.
[2] *Ibid.*, vol. vii (1885–86), p. 428.
[3] *Ibid.*, vol. xii (1890–91), p. 755.

accountant had joined hands in producing a book entitled "Factory Accounts[1]."

What Taylor did was to integrate and express as a series of principles, constituting the basis of a general philosophy, a whole range of ideas and experiments bearing on the ordering of industry which were coming to the surface in the last quarter of the last century as a logical result of the evolution of a mechanized economy. In the first flush of the industrial revolution, there was little time or effort available to consider such questions. The excitement of the new technical developments, the tremendous mastery over our material environment which they promised, absorbed all the inventiveness, initiative, and energy which were available. As Professor Bowie has observed, "Employers did not turn to operating profits until promotion profits began to dwindle."

Yet long before Taylor's time the nature of the problem which he faced had been recognized and the main outlines of his philosophy had been stated. In 1832, Charles Babbage, Lucasian Professor of Mathematics in the University of Cambridge, published the first edition of his "Economy of Manufactures." He was a man of scientific training and interests who had invented a calculating engine and in connexion with it had visited many factories, viewing their practice with a detached and analytic eye. He was, as he wrote, "insensibly led to apply to them those principles of generalisation to which my other pursuits had naturally given rise."

In 1835, Dr. Alexander Ure published his "Philosophy of Manufactures." He made the interesting point that Arkwright's contribution to textile practice was primarily as a manager rather than as an inventor. Thirty years before Arkwright, James Wyatt of Birmingham had invented and patented a practical "spinning engine without hands." But Wyatt was "of a gentle and passive spirit, little qualified to cope with the hardships of a new manufacturing enterprise." The main difficulty lay "in training human beings to renounce their desultory habits of work and to identify themselves with the unvarying regularity of the complex automaton. To devise and administer a successful code of factory discipline, suited to the necessities of factory diligence, was the Herculean enterprise, the noble achievement, of Arkwright."

In other words, technical progress is not enough. It creates, by its very achievements, acute human problems which must be solved if society is to reap the benefits which those achievements make possible. Or, in Herbert Spencer's words, "socially as well as individually, organisation is indispensable to growth; beyond a certain point there cannot be further growth without further organisation."

We cannot imagine, moreover, that the immense industrial successes recorded by this country in the first three quarters of the nineteenth century were registered without much systematic and orderly manage-

[1] "Factory Accounts," Garcke and Fells. Crosby Lockwood & Sons, London, 1887.

ment, with many improvements and innovations. The history is largely lost, though we have isolated examples, such as Mr. Roll's study of Messrs. Boulton and Watt's Soho factory [1]. But, on the whole, this management, however successful, was empirical and traditional. Men learned by "experience"; and the limitations of individual experience unstimulated by comparison with the methods of others, unfortified by reading, and uncorrected by reference to general principles, are obvious.

The work of British pioneers, such as Babbage and Ure, on the intellectual side, made little impression on the great mass of their countrymen. As a living historian has recorded in another connexion, "inability to apprehend general ideas appeared to stand between the people of England and their disturbing impact. . . . In Great Britain the pursuit of theory was left to professed theorists whilst an obstinately practical community eschewed the primrose path of general ideas and confined itself austerely to the solution of particular problems [2]." Moreover, employers were well insulated from any violent awareness of the social consequences of what they were doing by a hedonistic economics which attributed to the will of Providence the sufferings of the employed; "not even the most dismal economist supposed that an employer was a commodity [3]." Social concern was left to professed reformers, such as Shaftesbury and Owen.

But the problems pressed equally obstinately for solution, and by the last quarter of the century they had assumed a sufficiently concrete shape to rivet the attention of engineers previously confined strictly to their own technical field. Two main issues emerged—the economic and the human.

It became clear that any business enterprise, if it was to succeed, must be a unity. It was not sufficient for the engineer to do his work in one corner and the accountant to do his work in another. Their thought must be integrated. The engineer must pay more attention to costs, to the economic aspect of his work: the accountant must not be satisfied with book entries; his figures must correspond with realities. Equally it was useless to design the most perfect machines and processes if the human beings upon whom the engineer was dependent for his results were progressively less inclined to co-operate in securing them. Means must be found to overcome the "ca' canny", the deliberate or subconscious restriction of output which was endemic in the industrial system. Hence, the pioneer papers of Towne and Halsey.

The greatness of Taylor's achievement was that he brought to a focus these various ideas and tendencies. At first, experimentally and in relation to isolated problems, later with assurance and comprehensively, he showed that immense progress was possible if the empirical and traditional view of management could be abandoned and the whole question examined afresh, with the same detachment as an engineer brings to a

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1 Erich Roll, "An Early Experiment in Industrial Organization."
2 Philip Guedalla, "The Hundred Years," p. 64.
3 E. A. Filene, "Successful Living in this Machine Age."

problem of stresses or a chemist uses in analysing a new material. Just as in the machine-shop on the work of a single lathe hand he substituted detailed observation, analysis, measurements, precise instructions as to speed and feed and angle of cut for the varying standards of personal skill which each craftsman had formerly brought to the task, so in the management of the shop as a whole he sought to establish and to inculcate general principles based upon research, in place of the accidental, individual experience which had previously prevailed. He was an engineer looking consciously at management questions as an engineer, in place of regarding them primarily as personal or political issues. In short, he applied to the tasks of organization created by a machine economy the same intellectual approach which had made possible the immense advance in the physical sciences on which machine industry itself was based. And it is this angle of approach, this doctrine, which is of importance in his teaching—not any of the particular devices he developed by applying it to concrete situations.

Indeed, the attitude adopted by Taylor and his associates and imitators was often referred to popularly in the United States as " the engineering approach " to management. This description is open to objection on the ground that it compares human beings to machines, an accusation frequently brought against Taylor. We may perhaps reflect that some of the detachment and understanding which we apply to machinery would not always be out of place in dealing with human relations. No one who is not a fool attempts to generate activity in a stalled automobile by kissing its radiator or kicking its differential—processes too frequently applied in handling persons, disguised under such vague titles of approbation as " tact " and " firmness." But on the whole, mechanistic parallels are unsatisfactory ; human organization is a biological and psychological, not a mechanical problem.

The title officially applied to Taylor's ideas, namely " Scientific Management," has also led to misunderstanding. It implies, of course, that the task of management should be approached in the scientific temper and with due regard to the underlying sciences : it does not connote that management is itself a science. It is and remains an art. For this there are a number of reasons, the chief of which is that management deals primarily with human beings. And the sciences bearing on human beings are in many respects insufficiently developed to form a reliable, comprehensive, or exact guide to practical action.

Professor J. B. S. Haldane has estimated that " in another two or three centuries " the psychologists " will be beating the politicians at their own game and usurping their power, provided the politicians have left a civilization in which psychology can exist." He bases his estimate upon the time required for physics to lay the foundations of metallurgy, for biology to be of use to the animal-breeder, and for dietetics to cure more than it killed. " Psychology is about as much more complex than

biology as biology than physics." Moreover, "to study psychology before we understand the physiology of the brain is like trying to study physics without a knowledge of mathematics. . . . At the moment the physiology of the nervous system is being worked out with great speed [1]."

The same point was made by Dr. Cyril Burt with direct reference to experimental psychology:

> "Individual psychology is not a code of rules and principles to be mastered out of hand in the lecture room or laboratory. It is not an affair of text-book terminology or of a teachable technique. It is the product of worldly experience acting on an inborn interest, an enthusiasm for persons as persons in the old *nihil alienum* spirit. To take an unknown mind as it is, and delicately, one by one, to learn its chords and stops, 'to pluck the heart out of its mystery and sound it from its lowest note to the top of its compass,' is an art and not a science. The scientist may standardise the methods. To apply those methods, and appraise the results, demands the tact, the temperament, the sympathetic insight, of the genuine lover of strange souls [2]."

Management is individual psychology raised to the power of such number of persons as are affected by its acts. To conduct a shop, a golf club, a parish, a large-scale business, to command an Army, to govern a city or an empire, are fundamentally the same problem. They involve ordering the affairs of the undertaking so that the purpose or purposes with which it was conceived are identified with the complex human motives of those who participate in it, whether as producers or consumers, as civil servants or private citizens, in such a way that each can and does contribute the best of which he or she is capable to the common end. Certainly it is an art and not a science. It is an extremely difficult art. But, like the art of medicine, it is to-day one to which many underlying sciences have contributed a modicum of exact knowledge, and which can be practised most effectively and efficiently only by those who approach it in the temper of the true scientific worker.

Taylor himself had to do much propaganda, to make many arbitrary, and to that degree unscientific, statements to secure any attention for the "mental revolution" he was propounding. But his deep insight never allowed him to forget that the basis of exact knowledge on which he was working was inadequate to the task. Modern experimental psychology had scarcely reached the surface of public consciousness in his day. Yet we find him writing: "There is another type of scientific investigation which should receive special attention, namely, the accurate study of the motives which influence men."

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[1] "Business and Politics." "Possible Worlds," p. 185.

[2] Cyril Burt, "Mental Differences between Individuals." British Association. 1923.

In the same year in which Taylor died (1915), a great Frenchman published a short book "General and Industrial Administration," which was essentially a complement to his work. Henri Fayol was an engineer and metallurgist who for 30 years was President of a great coal and iron combine. He had been phenomenally successful. Towards the end of his life, he attempted to state in logical form the principles which underlay his skill as an administrator. Thus, whilst Taylor had driven his researches and experiments upwards from the individual worker to the causes which hampered him as a producer, Fayol applied the same scientific spirit downwards from the task of the leader as it ramified through and depended upon each succeeding level of supervision in the hierarchy. Both men dealt with the same central subject—management; both men used the same experimental material—the industrial enterprise. But they examined that material from opposite ends of the scale.

Fayol's "theory of administration" is less known in Great Britain than it should be; but it has had a profound influence in the Latin countries and is well known to all serious students of administration.

In the 35 odd years which have elapsed since, the lead given by these two men has been followed by an immense development in the technique of management, based upon the principles which they first enunciated. It has been an uneven development. No citadel is harder to storm than the walls of habit with which men surround their customary ways of working. No weapon is wielded more fiercely than the rationalizations with which they justify accepted thinking or vested interest. But in every industrial country in the world there are a proportion of factories in which a sincere attempt is made to manage in the scientific spirit; if this proportion is sometimes small, infinitely smaller is the number of undertakings in the economic field which have not been touched in some aspect of their management by the modern spirit. Moreover, research and experiment have not stopped in the workshop—Taylor's main field. They have been extended outwards to every function—advertising, marketing, selling, clerical work, accounting, buying, higher organization, and so on.

When we turn from industry to government, the pace is slower. You were perhaps surprised that I included commanding armies and governing empires among the activities to which Scientific Management is applicable. That brings me to two more names of individuals who have developed the work of Taylor and Fayol. Hundreds of other engineers and managers and research workers have, of course, contributed to an evolution which has been as swift and wide and pervasive as it has been, in some countries, loquacious. There are some twenty thousand volumes on management in the New York public library; on which fact a former Professor of English Literature in Cambridge University has written the perfect comment:

"The real tragedy of the Library at Alexandria was not that the

incendiaries burned immensely, but that they had neither the leisure nor the taste to discriminate [1]."

Walther Rathenau, Foreign Minister of the German Government in 1920, and the first victim of Hitlerite gangsterdom, was by training an electrical chemist. Before he was 40, he was Chairman or a leading Director of more than eighty companies in the electrical industry. In the last war he became Controller of Raw Materials and laid the broad foundations of the scheme which enabled his country to resist the Allied blockade for so long. It was impossible for such a man to think narrowly of the industrial problem. The spectacle of a maelstrom of Lilliputian enterprises bitterly assaulting each other with competitive bodkins seemed to him as little calculated to solve the world's economic difficulties as society's recent enthusiasm for tanks of minute tropical fish. As a first-class scientist and technician, he was fully appreciative of Taylor's work. He applied his concepts and method, however, both practically and in his writings, not to the individual undertaking, but to a whole industry and, after the war, as Minister of Reconstruction, to the whole structure of the German economy.

His own time was too short, and the times in which he worked too difficult, for any conclusive results to be achieved. But he started the rationalization movement in Germany which was endorsed by the World Economic Conference of 1927. Had any framework of political wisdom been strong enough to hold the European comity of nations together, he might well stand to-day as the forerunner of a new age rather than as the last flush of sanity before the nightmare of the Third Reich.

Mary Parker Follett was a Boston spinster who devoted a modest competence and one of the clearest brains of our generation, first to social work in her native city, parallel with that of the study of political science, and finally to work in business administration. As a political scientist, her two books, "The New State" and "Creative Experience" struck a new note; they earned the unqualified praise and enthusiastic friendship of the best minds on the subject in this country: for instance, the late Lord Haldane of Cloan, Professor Harold Laski, and Sir Arthur Salter.

Her reasons for pursuing the study of government down the avenue of business management are best given in her own words:

> ". . . it is among business men (not all, but a few) that I find the greatest vitality of thinking to-day and I like to do my thinking where it is most alive. . . . I find so many business men who are willing to try experiments. . . . *There* men are not theorising or dogmatising, they are thinking of what they have actually done and they are willing to try new ways next morning so to speak. . . . Whatever problems we solve in business management may help towards the solution of world

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[1] Sir Arthur Quiller Couch, "On the Art of Reading," p. 12.

problems, since the principles of organisation and management which are discovered as best for business can be applied to government or international relations."

It will be seen that there was no intellectual break. Her subject was government. She found the greatest volume of practical experiment and freedom of thought about government in business. So, though she had no business experience, she immediately turned her inquiring and exceptionally lucid mind on to the problem. Her collected papers in this field, recently published under the title "Dynamic Administration," are a textbook for the twentieth century on how to make democracy effective.

Finally, in 1936, Franklin D. Roosevelt, who is at present occupying for a third term the greatest elective office in the world, found himself troubled with mal-organization in the executive branch of the Federal Government of the United States—the central authority controlling a nation of 140 million people. To advise him he appointed a committee of three men, all of them specialists in modern methods of administration. The committee appointed its own independent research staff of twenty-eight persons under a qualified director. Their Report—the Report of the President's Committee on Administrative Management—is a short but complete blue-print for the reorganization of the Departments of State and the numerous subsidiary authorities, and the creation of the necessary machinery to enable the President, as the Chief Executive Officer of the nation, to co-ordinate their activities effectively. It is a classic state-document, shot through and through with the concepts, methods, and principles of scientific management.

The President, in his letter submitting the Report to Congress, left no doubt as to his own views:

> "In these troubled years of world history a self-government cannot long survive unless that government is an effective and efficient agency to serve mankind and carry out the will of the nation. A government without good management is a house builded on sand. . . . In striving together to make our government more efficient, you and I are taking up in our generation the battle to preserve that freedom of self-government which our forefathers fought to establish and hand down to us. They struggled against tyranny, against non-representative controls, against government by birth, wealth or class, against sectionalism. Our struggle now is against confusion, against ineffectiveness, against waste, against inefficiency. This battle, too, must be won, unless it is to be said that in our generation national self-government broke down and was frittered away in bad management."

It seems a far cry from Taylor, the unknown engineer struggling with inefficiencies in the lathe shop at Midvale, to the greatest President of one of the world's greatest democracies, one of the triumvirate who lead a

host of allied nations fighting for freedom, struggling with the limitations of bureaucracy. And yet, as the Report says :

> "The foundations of effective management in public affairs, no less than in private, are well known. They have emerged universally wherever men have worked together for some common purpose."

Less than half a century divides Taylor's "A Piece-Rate System" from that application of his thought to human organization on the largest scale. It is an amazing justification of his life's work.

No one is a greater nuisance in the academic world than the enthusiast for a special subject who cannot rest till he has added that subject to an already overcrowded curriculum. But for a number of reasons the study of modern methods of management has a special claim to be considered in any engineering syllabus, not so much as one more technical subject, but rather as the pivot of that "Humanistic side of engineering education" which, as Professor Inglis has emphasized, "must relate mainly to the present and the future."

The utilitarian point of view, though important, is not the most important aspect of the subject. I have not the figures for Great Britain, but a very comprehensive inquiry into engineering education in the United States, conducted between 1923 and 1929, showed that whilst "young engineers in three cases out of four begin their careers in the realm of technical duties, by the age of 40, three out of five are occupied with administrative duties." From general observation, that conclusion would appear to be broadly true of Great Britain and to be largely unaffected by the particular type of engineering involved. Civil engineers engaged in constructional work of all kinds have always been large employers of labour; and municipal employment usually involves the successful handling of committees and other aspects of administration.

It is, of course, highly undesirable that all the best brains in any profession should be drawn off from technical and research work into the administrative field. But in any organized industrial undertaking, the number of senior and highly-paid posts on the research side is likely to be smaller in proportion than the number of administrative posts of equal value for which a qualified engineer with experience of that industry would be a strong candidate. In any event, the only alternatives to the engineer-administrator are either to draw administrators from some other technical background, such as the law or accountancy, or to develop a special class of administrators to whom engineers will serve as technical advisers.

The first alternative has the disadvantage that the two disciplines mentioned are apt to influence their graduates towards thinking in words or figures, rather than in realities. In any event, the personal qualities required for leadership are so rare and in our complex modern organizations the demand for leadership at all levels is so great, that every profession

must contribute its share of talent. The second alternative is that adopted in the Civil Service, with its special "Administrative Grade." From recent observation, it is not one likely to provide either a supply of the type of leaders required for large undertakings demanding commercial or constructive ability and initiative, nor one calculated to develop the best type of technical originality and energy among engineers associated with it.

It has frequently been urged that since the engineering graduate will not rise to a position of managerial responsibility for some years after entering practical life, introduction to management principles should be the business of his subsequent employer. But this view omits the very important considerations that the development of a sound method of approach to any subject is essentially a matter of basic education and that, save in very exceptional cases, the employer is bound to be more interested in the immediate utility of any particular employee than in a long-period plan for the development of suitable leaders for the future. In my own experience it is still rare to find high administrators, whether in the public service or in private employment, who escape the ordinary human dislike of contemplating a world relieved of their unique personal contribution. The subject of training those who are to succeed them is neither very popular nor one that they are inclined to face realistically. The elective bodies common in central and local democratic government are apt to take even shorter views.

It therefore seems desirable, on purely practical grounds, that students before graduation should have an opportunity of acquainting themselves with the broad outlines of modern thought on business management. They must not expect to spring from the examination room armed cap-à-pie as high-powered executives. They can expect to have grasped the way in which their engineering knowledge and training can be applied to this somewhat different field and, *more important*, the directions in which it should be supplemented by further study of the subject and modified by a reasoned modesty. They should, too, have done something to bridge the gap which must exist between any system of education and its application to practical life. They should apprehend more quickly and with fewer misunderstandings the meaning and purpose of any organization in which they find themselves, so that what must appear at first sight a welter of confusing detail takes shape with the minimum lapse of time. They should be better qualified at an earlier date to act as an "assistant to" a responsible executive, whether on the managerial or technical side, and such a position is far the best opportunity open to the young engineer to equip himself for more serious responsibilities.

For all these reasons, some training in management before graduation seems to me desirable and likely to be of great practical advantage to engineering candidates, provided always that they do not imagine that they are learning to manage. That can be learned only in practice, just

as commanding armies and winning battles with them can be learned only in the school of actual fighting—never from text-books on tactics and military history. But, following out the analogy, the officer who has studied his texts and made them a part of his thought, is far more likely to benefit from the practical experience of war which comes his way, than the officer who has preferred polo as a preliminary exercise.

This conclusion is borne out by the figures collected in the United States in connexion with the Report on Engineering Education already mentioned. Of some thousands of engineering graduates who were asked what subjects should receive more attention in their curriculum, 41·4 per cent. replied " Commercial, Business and Economic subjects," and a further 11·5 per cent. replied " Industrial Law, Salesmanship, Management, or Accounting." It is also of interest to note, in connexion with a further point that I shall make below, that another 26·3 per cent. mentioned English.

There are, however, deeper reasons why I should recommend a course in management as part of any engineering curriculum. As Professor Inglis has said, " Engineering is now shaping the destiny of civilization." But the men and women who labour in the factory, the forge, and the mine are not always amenable to the shaping, or enthusiastic about the destiny. For this there is a definite reason, on which some of the most recent researches in management, notably those conducted under the direction of Professor Elton Mayo, of Harvard, on working teams at the Western Electric Company, are beginning to shed some light.

Dr. North Whitehead, one of Professor Mayo's collaborators, has perhaps put the point most clearly. He emphasizes the importance of primitive leadership where " the leader's function is such as to assist the group in maintaining its customs, its purposes, and its attitudes undamaged by the chance ineptitudes of the less experienced or less skilful members [1]." However unrealized, the importance of this social integrity to the small working group is enormous for " ultimately all worth-while living is conceived as directed activity within an accustomed social pattern [2]." Thus, " the outstanding characteristic of the mechanic leader is his intense pride in and unswerving loyalty to the detailed procedure by which he exercises his skill. Any attempt on the part of his ' boss ' to modify these by one hair's breadth will evoke the most unmeasured anger. It is as though a brutal assault had been made on his household gods. And that is exactly what it amounts to [3]."

" From the point of view of a modern code of values this leadership has one capital defect: it is not progressive [4]." But the engineers, and their employers, have " deliberately organized logical thinking in such a

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[1] T. N. Whitehead, " Leadership in a Free Society," p. 70.
[2] *Ibid.*, p. 61. [3] *Ibid.*, p. 70. [4] *Ibid.*, p. 75.

way as to lead to a stream of improvements [1]." And "management is prone to rearrange the working conditions of its employees with scant regard for the social routines and sentiments it is unwittingly breaking [2]." "The modern leader is no longer quite a member of his group, working by their side and sharing their daily lives. . . . Whether he realises the fact or not, he is in danger of directing a formed society from without; a society that will evolve defense mechanisms and sentiments of antagonism if its social living appears to be in danger of interruption [3]."

The same point, though without the experimental background, was noted by the authors of the Garton Memorandum published in 1916, as a criticism of Scientific Management itself. They attributed much of labour's resistance to "the worker's instinctive aversion . . . to performing without variation a cycle of mechanical movements . . . prescribed not by himself or by the traditions or master-craftsmen of his class, but by an outside and unsympathetic authority in the shape of the scientific expert [4]." This is an interesting illustration of the danger—already emphasized—of treating Scientific Management as a fixed body of doctrine rather than as a mental attitude, a method of approach. Here we have the latest research work in the management field emphasizing a danger which, twenty years before, had been charged against the very spirit of inquiry which has led to the later thinking.

It again emphasizes that technical skill and achievement are not enough. As Ure wrote of Arkwright, the main difficulty lies "in training human beings to identify themselves with the complex automaton." And it is seen to be a difficulty involving an understanding not only of individual psychology, which if we follow Haldane and Burt, is far from being an exact science or a safe guide to action, but also of social psychology and modern anthropology. From the social standpoint, the engineer who may become a manager should learn to be something more than an engineer.

At the same time, his engineering training in itself will be of immense value to him in this difficult social field. As Westaway has pointed out:

> "Science has one immense advantage over all other subjects. All facts can be obtained at first hand and without resort to authority. The learner is thus put in a position of being able to reason with an entirely unprejudiced mind. It is this possibility of *self-elimination in forming a judgement* that must be regarded as the greatest possible specific result of scientific teaching [5].

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[1] T. N. Whitehead, *loc. cit.*, p. 59.

[2] *Ibid.*, p. 59.

[3] *Ibid.*, p. 79.

[4] Garton Foundation, "Memorandum on the Industrial Situation after the War." London, 1916.

[5] F. W. Westaway, "Scientific Method," p. 6.

If you turn to Mary Follett and her studies of organization, you will find that she appeals again to this *self-elimination*, not only in forming a judgement, but also in giving orders, in participating in discussion, in leading other people, as one of the main methods of avoiding the sentiments of antagonism which Whitehead properly emphasizes. If any group of people can be made to understand that what you are asking them to do is to obey, not your personal authority, but a series of facts—in Miss Follett's famous phrase, "The law of the situation"—you have gone a long way towards reconciling them to necessary change. The mere process of expounding and discussing the facts is a social process identifying the specialist with the group whose social life he is disturbing.

Like all things, however, its utility will be greatly enhanced if its limitations are also appreciated. We must remember that individual psychology, and much less social psychology and anthropology, are not exact sciences: they are tools to be used with great discretion. As Westaway observes in the passage already quoted:

> "A classical training (by which he means a humanistic training) is admittedly productive of one great advantage over a training in science, and that is in the power it confers *in the balancing of probabilities in which the human element is dominant* [1]."

The chief danger facing the student of engineering who has the imagination to appreciate both the opportunity and the responsibility which face him, is that he will try to eat too largely of the tree of knowledge: and here again I would quote Sir Arthur Quiller Couch lecturing in this University:

> "Against knowledge, I have, as the light cynic observed of a certain lady's past, only one serious objection—that there is so much of it. . . . If you crave for knowledge, the banquet of knowledge grows and groans on the board until the finer appetite sickens. If, still putting all your trust in knowledge, you try to dodge the difficulty by specialising, you produce a brain bulging out inordinately on one side, on the other cut down flat and mostly paralytic at that: and in short, so long as I hold that the Creator has an idea of a man, so long shall I be sure that no uneven specialist realises it [2]."

To become a Manager you must first become a man: and to become an educated man, there must be some content of humanism in your mental make-up.

I concur with Professor Inglis, that "that form of culture known as *literae humaniores* is a luxury which, in the case of an engineering student, must be left to individual enterprise [3]." But I believe that a

[1] F. W. Westaway, *loc. cit.*, p. 6.

[2] Sir Arthur Quiller Couch, "On the Art of Reading," p. 12.

[3] Journal Inst. C.E., vol. 17 (Session 1941–42), p. 9 (Nov. 1941).

solution to this difficulty may be found in a noble paragraph of the famous Report of the Newbolt Committee on "The Teaching of English in England"—surely in itself one of the best-written documents ever issued by His Majesty's Government:

> "In the course of our enquiry it has been borne in upon us time and again that our education is too remote from life. . . . Hitherto the best currents of educational thought and experiment, deriving from the time of the Renaissance, have either despised or ignored the commercial facts of the modern world. The result has been a cleavage disastrous both for education and industry. . . . We claim . . . that an English humanism including the study of literature and of the language as an instrument of thought and expression, if made actual by being brought closely into touch with the main preoccupations of the students, might go far not only to ennoble . . . education . . . but also to bridge the gulf between industry and culture."

Now management—Scientific Management—as I have sketched it to you to-night, is a subject which lends itself to the use of spoken and written English. Though Heaven forfend that you should take some of the books which have been written on the matter for anything but awful examples of what may happen when engineers lack any cultural background. Certainly both in its origins and in its bearing upon their future, it is closely in touch with the main preoccupation of engineering students. If I may make so bold as to hazard a suggestion in a field in which I have no direct responsibility, it is this—that parallel with your technical studies, which should still form the outstandingly greater proportion of your curriculum, a certain portion of your time should be devoted to English—and compulsorily devoted to English. But the subjects in which you practise the written and spoken word should be economics, the evolution of industry, the structure of central and local government, management thought, and the history of the underlying sciences upon which the engineering and management techniques are based, including the lives of the great pioneers.

I conceive of these subjects in the very broadest sense—not as an effort to inculcate knowledge, but as an "opening of the windows of the mind" on the immense, the undreamed of possibilities of human well-being which are in your hands if you use them aright. In a phrase, what I am aiming at is a humanism for the twentieth century, based upon the accumulated treasure of all our yesterdays:

> ". . . Shakespeare's hand,
> Milton's faith and Wordsworth's trust
> In this our chosen and changeless land."

but looking to the future—its content and purpose the making of good engineers who are also good citizens of a scientific age.

To quote once more, in conclusion, from the Wickenden Report:

"It is one thing to determine how the engineer should be trained to do the things which tradition and past practices have prescribed as his normal work. It is quite another thing to picture his place in the New Epoch which power and machinery, science and engineering and productive industry is initiating [1]. . . ."

The engineer of to-morrow will not rise to leadership by abandoning his distinctive role or by permitting it to become ill-defined. He will do so by remaining essentially an engineer, by becoming a more competent engineer, by extending the reach of engineering methods and ideals to larger realms of life, and withal, by making himself a team-mate eagerly desired by other types of men. If engineering education is to serve these ends, it must safeguard all the distinctive qualities and virtues of its past and add to them a more generous humanism [2]."

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[1] "Engineering Education." Society for the Promotion of Engineering Education, U.S.A., 1920, vol. i, p. 11.

[2] *Ibid.*, p. 53.

OBITUARY.

JOHN SWITZER OWENS was born at Enniscorthy, Co. Wexford, on the 28th July 1871, and died in hospital at Cheam, Surrey, on the 6th December 1941. At the age of 16 he entered Trinity College, Dublin, and studied medicine, leaving in 1892 with the qualifications of B.A., M.B., and B.Ch. In 1896 he obtained the degree of M.D. Five years later, and owing to a great liking for engineering and no great liking for medicine, he decided to give up the latter and become an engineer. After nearly two years of training in pattern making and foundry work at Gateshead-on-Tyne he was transferred to the works of the Wallsend Slipway and Engineering Company. He also studied at the Durham College of Science, and in 1899 commenced a pupilage under the late Mr. Edward Case, Assoc. M. Inst. C.E., becoming later an assistant to the firm of Case and Gray on sewage-disposal works, reclamation, and foreshore protection works, and collaborating with Mr. G. O. Case (son of the late Mr. Edward Case) in a text-book on "Coast Erosion and Foreshire Protection." After five years he commenced private practice with Mr. G. O. Case in Westminster and acted as consulting engineer for sea works to the East Sussex County Council. He also designed plant for concrete-mixing, dredging, and the drying of materials. In 1912 he was appointed consulting engineer to the San Domingo mines, Portugal, and in 1929 he received a similar appointment to the Rio Tinto copper mines in Spain, for which he designed new equipment including subaqueous drilling plant, air lift pumps, and an alignment-indicator for boreholes. He also acted as technical adviser to the London Smoke Abatement Society, and made extensive investigations on atmospheric pollution, serving under the Meteorological Office from 1917 and under the Department of Scientific and Industrial Research from 1927, when he was appointed Superintendent of Observations, an office he retained until his death. He designed many instruments for the investigation of pollution problems and was joint author, with Sir Napier Shaw, of a book on "The Smoke Problems of Great Cities," published in 1926, whilst many of his studies formed the subjects of Papers presented to technical institutions.

Dr. Owens was elected an Associate Member of The Institution on the 8th April 1902. In 1912 he was joint author, with Mr. F. J. Wood, formerly Assoc. M. Inst. C.E., of a Paper entitled "Reinforced Concrete Sea Defences[1]."

In 1896 he married Kate Cordelia Brunskill, daughter of the late Edward Brunskill, and had no children.

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[1] Min. Proc. Inst. C.E., vol. clxxxix (Session 1911–12, Part 3), p. 292.

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NOTE.—*Pages* [1] *to* [22] *can be omitted when the Journal is bound in volume form.*

NOTICES

No. 5, 1941—42

MARCH, 1942

MEETINGS, SESSION 1941—42.

ORDINARY MEETING.

The following Paper will be discussed on the date shown :—

Apr. 14 (Tues.). (2 p.m.) *† "**Post-War Planning and Reconstruction**", by H. J. B. Manzoni, C.B.E., M. Inst. C.E.

ROAD ENGINEERING SECTION.

Apr. 21 (Tues.). (5.30 p.m.) † Paper for discussion : "**The Influence of Modern Road Lay-out on Bridge Design**", by C. S. Chettoe, B.Sc., M. Inst. C.E.

RAILWAY ENGINEERING SECTION.

Apr. 28 (Tues.). (5.30 p.m.) † Paper for discussion : "**The Repair of War Damage to Railway Way and Works in the London Area 1940 and 1941**", by Arthur Dean, M.Sc., Assoc. M. Inst. C.E.

Members of the Railway Engineering Section are notified that a ballot for the election of new members of the Committee will be taken immediately preceding the meeting to be held on Tuesday, 28 April. Nominations for the Committee should be sent in writing to the Secretary of The Institution not later than Monday, 13 April. The present serving members of the Committee (appointed provisionally) are Messrs. R. Carpmael (Chairman) ; F. E. Wentworth-Sheilds ; W. T. Halcrow ; R. J. M. Inglis ; V. A. M. Robertson ; George Ellson ; and W. K. Wallace.

SPECIAL JOINT MEETING.

The Council of The Institution have decided, in conjunction with the Councils of the Institutions of Mechanical and Electrical Engineers, that a special joint meeting of the three Institutions should be held to discuss

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* A brief synopsis of the Paper appears at p. [14], *post.*

† Advance proofs, for those who intend to be present, will be available about a fortnight before the meeting, and copies may be obtained upon application to the Secretary.

" The Application of Statistical Control to the Quality of Materials and Manufactured Products " as an immediate development which would be of benefit in the output of many forms of munitions of war.

This special joint meeting has now been arranged to take place in the Great Hall of The Institution of Civil Engineers at 2.30 p.m. on Wednesday, 15 April. The Chair will be taken by the President of the Institution of Electrical Engineers, Sir Noel Ashbridge, and the subject for discussion will be introduced by Dr. C. G. Darwin, M.C., M.A., F.R.S., and Sir Frank Gill, K.C.M.G.

For some 15 years there has been developed in this country, and at the same time independently in the United States of America, a new technique for handling problems concerning the control of the quality of manufactured products by the application of statistical methods. These methods have progressed so far beyond the laboratory or experimental stage that they are available for immediate application in such a simple form that it is claimed that there will be no fall in output during their initiation, and that within a remarkably short period either an improvement in average quality or an increase in the quantity of the accepted product will result. Results in factories in which statistical control has been applied during the last 12 months support these claims.

The theory underlying the method is described in B.S. 600/1935 of which a few copies are still available from the British Standards Institution (price 5*s*.). The practical application of the method will be found in concise form in B.S. 1008/1942, which is about to be published (price 3*s*. 6*d*.), and also in a new edition of B.S. 600, which will be issued shortly. In an abbreviated form the opening speakers' remarks will be obtainable in advance of the meeting from the Secretaries of the Institutions, to whom application should be made for tickets of admission. In the case of members of The Institution of Civil Engineers applications should, of course, be addressed to the Secretary of that Institution.

SPECIAL ANNOUNCEMENTS.

MILITARY SERVICE.

POSTING OF STUDENTS OF THE INSTITUTION TO THE ROYAL ENGINEERS.

Referring to the announcement on p. [4] of the " Notices " Section of the January Number of the Journal, the following is to be observed :—

(*a*) Nothing in that announcement applies to *serving* Members and Students, who must apply through the correct military channels.

(*b*) With regard to the third paragraph of the announcement (" If, however, a Student . . . at once "), it must be emphasized

that in the case where a member or Student receives a calling-up notice he should report the fact to the War Office (A.G.7), *before* the date on which he has to report in compliance with such calling-up notice.

GENERAL ANNOUNCEMENTS.

THE JOURNAL.

The next Number of the Journal will be published on the 16th April.

"INGENUITY" COMPETITION.

Papers are invited from Corporate Members and Students in competition for a Prize of Twenty-five Guineas to be awarded by the Council for a description of an engineering problem and the method adopted to solve it.

The article should not exceed 2,000 words, and must describe a specific problem involving immediate action and ingenuity displayed in meeting it. The problem must have arisen in the competitor's own experience, and the action taken must have been to some extent—not necessarily wholly—his own idea. These facts must be vouched for in a satisfactory manner.

The Papers should reach the Institution by the 30th April 1942, with the MS. marked "Ingenuity" Competition in the top left-hand corner of the first page.

The Council reserve the right to publish the winning entry, or any other selected entries, and should such entries relate to engineering problems arising out of the war, The Institution would submit them to the Censor for permission to publish.

BRITISH STANDARDS INSTITUTION.

The Council have nominated Mr. J. R. Beard, M. Inst. C.E., as the representative of The Institution and the Founder Institutions of the original Engineering Standards Committee, on the Executive Committee of the British Standards Institution.

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH.

AERODROME ABSTRACTS.

The Road Research Laboratory of the Department of Scientific and Industrial Research has kindly given permission to The Institution to reprint "Aerodrome Abstracts" in the Journal and Nos. 1–13 appear at pp. [15]–[21], *post*.

TRANSFERS, ELECTIONS, AND ADMISSIONS.

Since 13 January 1942, the Council have transferred six Associate Members to the class of Members, elected one Honorary Member, two Members and eleven Associate Members, and have admitted thirty-eight Students.

DEATHS AND RESIGNATIONS.

Deaths.

Cookson, Alfred Chorley	*Member*
Cowan, Edward Woodrowe	,,
Ede, Cecil Burton	,,
Ford, Hugo Robert	,,
Hooley, Edgar Purnell	,,
Laurie, Gordon Colet	,,
Money-Kent, Julian Money Vernon	,,
Pettigrew, William Frank	,,
Thomson, Gilbert	,,
Archibald, Robert Douglas, B.Sc.	*Associate Member.*
Carne, Frederick William	,, ,,
*Brockbank, Robert	*Student.*

* *On Active Service.*

Resignations.

Alexander, James Maclean	*Member.*
Given, Ernest Cranston	,,
Matthews, Thomas Leigh	,,
Robinson, Leonard Leslie	,,
Thom, Gordon William, M.C.E.	,,
Adams, Frederick Guy Trevenen	*Associate Member.*
Boissiere, Hector Elie de, B.Sc.	,, ,,
Brown, William Rochester	,, ,,
Buddle, Geoffrey Armstrong, D.S.O., M.C., B.Sc.	,, ,,
Chamberlain, William Young	,, ,,
Edwards, Lindsay Ernest	,, ,,
Kidston, Malcolm Glen, B.Sc.	,, ,,
Liversedge, John William	,, ,,
Nanson, Gerald Leighton, B.E.	,, ,,
Rogers, Arthur	,, ,,
Searle, Basil James	,, ,,
Smith, Robert Melville	,, ,,
Thomson, Hedley Jeffreys	,, ,,
Waithman, Charles Harold	,, ,,
Walker, George Dutton	,, ,,
Walton, Eric Bell	,, ,,
Watts, Stafford Tracey	,, ,,
Canning, Ronald Maurice	*Student.*

A SELECTIVE LIST OF RECENT ADDITIONS TO THE LIBRARY.

[Journals, Proceedings of Societies, etc., are not included.]

Agricultural Machinery. Smith, H. S. "Farm Machinery and Equipment." 1937. McGraw-Hill. 18s.

AIRCRAFT. BOGGESS, H. E. "Aircraft Sheet Metal Work." 1941. Pitman. 7*s*. 6*d*.

—— IRVIN, G. E. "Aircraft Instruments." 1941. McGraw-Hill. 35*s*.

BEAMS. SHEPLEY, E. "Continuous Beam Structures." 1942. Concrete Publications. 7*s*. 6*d*.

CORROSION. *See* METALS.

ELECTRICITY—MOTORS. VAN BRUNT, G. A., and ROE, A. C. "Winding A.C. Motor Coils." 1938. McGraw-Hill. 21*s*.

ELECTRICITY—TRANSFORMERS. AVERY, A. H. "Small Transformers." 1941. Marshall. 1*s*.

ELECTRODE POTENTIAL BEHAVIOUR. *See* METALS.

GEOPHYSICS. JAKOSKY, J. J. "Exploration Geophysics." 1940. Times Mirror Press, Los Angeles. 36*s*.

IRON AND STEEL. BRITISH CAST IRON RESEARCH ASSOCIATION. "Sampling and Chemical Analysis of Cast Ferrous Metals." By E. T. Austin. (Special Publication No. 7.) 1941. The Association, Birmingham. 15*s*.

MARINE ORGANISMS. MARITIME SERVICES BOARD OF NEW SOUTH WALES. "Destruction of Timber by Marine Organisms in the Port of Sydney." By R. A. Johnson and F. A. McNeill. (Supplementary Report No. 2.) 1941. The Board, Circular Quay, Sydney. No price.

MATHEMATICS. CARSLAW, H. S., and JAEGER, J. C. "Operational Methods in Applied Mathematics." 1941. Clarendon Press. 17*s*. 6*d*.

METALS. GATTY, O., and SPOONER, E. C. R. "Electrode Potential Behaviour of Corroding Metals in Aqueous Solutions." 1938. Clarendon Press. 25*s*.

*MINES AND MINING. PEELE, R., and CHURCH, J. A., *Ed.* "Mining Engineers Handbook." 3rd ed. 1941. 2 vols. Chapman & Hall. £4 10*s*.

PHOTOELASTICITY. FROCHT, M. M. "Photoelasticity." Vol. 1. 1941. Chapman & Hall. 36*s*.

RADIO-FREQUENCY MEASUREMENT. HARTSHORN, L. "Radio-Frequency Measurements by Bridge and Resonance Methods." 1941. Chapman & Hall. 21*s*.

SHEET METAL. *See* AIRCRAFT.

SHELLS. WILSON, W. M., and OLSEN, E. D. "Tests of Cylindrical Shells." 1941.

—— SKEW SLABS. JENSEN, V. P. "Analysis of Skew Slabs." 1941. Univ. of Illinois Eng. Exp. Stn. Bulletins 331 and 332. 1 dollar each.

SOLIDS. SEITZ, F. "Modern Theory of Solids." 1940. McGraw-Hill. 49*s*.

TRADE WASTES. DEPT. SCI. & IND. RES. Water Pollution Research Technical Paper No. 8. "Treatment and Disposal of Waste Waters from Dairies and Milk Product Factories." 1941. H.M.S.O. 4*s*.

VIBRATION. DEN HARTOG, J. P. "Mechanical Vibrations." 2nd ed. 1940. McGraw-Hill. 35*s*.

—— WILSON, W. K. "Practical Solution of Torsional Vibration Problems." 2nd ed. 2 vols. 1940 and 1941. Chapman & Hall. 84*s*.

*WATER-MILLS. MILLER, W. T. "The Water-Mills of Sheffield." 1936. Issued privately by Author. No price.

(* *The foregoing books, with the exception of those marked with an asterisk, may be borrowed from the Loan Library.*)

LOCAL ASSOCIATIONS.

The following arrangements have been made for forthcoming meetings of the Local Associations. The arrangements are in the hands of the Committees of the Associations concerned and all communications respecting them should be addressed to the respective Honorary Secretaries :—

NORTHERN IRELAND ASSOCIATION.

Apr. 27. Annual General Meeting.

NORTH-WESTERN ASSOCIATION.

Mar. 21. "Soil Mechanics and Site Exploration", by L. F. Cooling, M.Sc.

Apr. 25. "The Engineer's Part in Town Planning", by H. J. B. Manzoni, C.B.E., M. Inst. C.E.

SOUTH WALES AND MONMOUTHSHIRE ASSOCIATION.

Apr. 18. Annual General Meeting (at Cardiff).

SOUTHERN ASSOCIATION.

Mar. 21. Joint meeting with the Southern Branch of the Institution of Mechanical Engineers. The Thomas Hawksley Lecture on "A Century of Tunnelling", by W. T. Halcrow (at Portsmouth).

Apr. 11. "Soil Mechanics and Site Exploration", by L. F. Cooling, M.Sc. (at Southampton).

Apr. 25. "The Analysis of some Engineering Problems associated with Clay Soils", by A. W. Skempton, M.Sc., Assoc. M. Inst. C.E. (at Portsmouth).

YORKSHIRE ASSOCIATION.

Apr. 11. "Shell Manufacture", by John Baker, M. Inst. C.E. (at Leeds).

REPORTS.

Bristol and District Association.

On Thursday, 5 February, at Bristol, Mr. J. Nelson Meredith, F.R.I.B.A., read a Paper on "The Replanning of Bristol"; and on Thursday, 12 February, at Gloucester, the film illustrating the Failure of the Tacoma Narrows Suspension Bridge was exhibited.

Edinburgh and District Association.

On Wednesday, 11 February, a Paper on "Some Foundation Problems and Soil Action" was read by Mr. A. R. Pollard, B.A., M. Inst. C.E.

Glasgow and District Association.

The film illustrating the Failure of the Tacoma Narrows Suspension Bridge was shown at a meeting held on Friday, 30 January.

North-Western Association.

On Saturday, 14 February, a Paper on "Recent Quay Construction at Newcastle-upon-Tyne" was read by Mr. J. R. d'O. Lees, M. Inst. C.E.

Southern Association.

On Saturday, 14 February, at Brighton, the Chairman, Mr. J. Parkin, O.B.E., M. Inst. C.E., gave an address on "Problems for the Municipal Engineer during and after a Blitz."

Yorkshire Association.

On Saturday, 24 January, at Leeds, Mr. J. M. Collie read a Paper on "Lessons of the Sheffield Blitz"; and on Saturday, 7 February, at a joint meeting at Leeds with the Yorkshire Branch of the Institution of Structural Engineers, Dr. Oscar Faber, O.B.E., M. Inst. C.E., read a Paper on "The Aesthetic Design of Engineering Structures."

LOCAL ASSOCIATIONS.

NOTE ON

"Lessons of the Sheffield Blitz."

(*Abridged.*)

Read by Mr. J. M. Collie, City Engineer and Surveyor of Sheffield, before a meeting of the Yorkshire Association, on 24 January 1942.

During December 1940, Sheffield experienced an air raid of blitz intensity which lasted some hours. A few days later a further raid, lasting several hours, started at approximately the same time. Both raids were severe, with continuous bombing. The usual blitz tactics were adopted: the initial planes dropped incendiary bombs across the city, followed by high-explosive bombs.

In the provision of Rest Centres and feeding facilities, £130,000 worth of materials and equipment was purchased in 3 days.

Organization.

Sheffield has a population of just over half a million and covers an area of 40,000 acres. There are 660 miles of repairable roads.

The Civil Defence services are organized in divisions, at each of which

there is a Report and Control Centre. There is also a Headquarters Control. The City Engineer is responsible for the rescue and decontamination services, the demolition of dangerous buildings and clearance of streets, and the repair of damage to roads and sewers, tramway permanent way, and bridges. The City Engineer's department is organized on a direct labour basis, with the necessary transport and plant.

The rescue service is officered by the City Engineer's technical staff on a voluntary basis. Officers do night duty on rota at all Control Centres, and there is a rota for Depot and Incident Engineers. Also, a special Highways Control is manned nightly, and deals with all incidents involving repairs to highways, sewers, bridges, and tramway-tracks.

There are rescue and decontamination depots, at each of which is a cleansing-station. Only half of these depots are regularly manned, the others being manned by volunteer parties. The organization is such that it can be extended to cover all depots in the event of heavy raids.

To a large extent, the rescue service has been recruited from the personnel of the City Engineer's department, most of whom are Corporation employees, and the rest outside volunteers.

All parties are dually trained in rescue and decontamination work; there is no separate allocation of personnel for decontamination purposes. A rescue school was established in Sheffield early in the War, and all personnel has been trained there. The School is now acting as a Regional School for training in rescue and decontamination.

Members of the City Engineer's staff have been trained in Callender-Hamilton unit bridge construction, and certain contractors designated by the Ministry of War Transport were at the call of Sheffield in the event of need.

In substitution for the foregoing arrangement, six of the full-time rescue squads have been trained in the erection of emergency bridging materials, and this system has also been approved by the Ministry of War Transport.

Rescue Service.

At the commencement of the first raid referred to, rescue squads were on standby duty in the City, and volunteers reporting during the night increased their number. The volunteer response was not so good as expected. Regional assistance was also given.

Tools.—There was very little demand for the more elaborate heavy-duty rescue equipment; the tools most in demand were shovels, crowbars, and debris-baskets.

The chief difficulty experienced in rescue work was making certain that all persons resident in a house had been accounted for. To a great extent, this difficulty arose in consequence of people being evacuated and having left the neighbourhood of the incident.

Lessons.—The rescue establishment was much too small. It was increased by 50 per cent. immediately after the first raid, and has since been increased by volunteer parties reporting on rota or sirens.

Services must be prepared to carry on without Incident Officers in a big raid. Incident Officers were not available.

With Regional approval, a substantial reserve of rescue personnel has been supplied by the formation of wardens' Supplementary Rescue Parties. Parties—one per group—have been formed and equipped with the usual small rescue tools, and their personnel has been trained by the permanent Rescue Instructors. The intention is that these Wardens' Parties will tackle any rescue job immediately. They will come under Report Centre control and the incident will be reported in the normal way, with the additional information that the Wardens' Party is attending to rescue. This message will be followed as soon as possible by a supplementary message which will state :—

(1) That the wardens have completed, or can complete, the job satisfactorily ; or,

(2) That the job is too big for them and that assistance is required from the regular rescue service.

A definite and foolproof system of reporting must be adopted in respect of assistance by means of Regional Parties.

The provision of definite information by wardens as to persons resident in a house is the most important lesson. Wardens' house-cards should contain particulars of the residents, where they normally shelter, the type and place of shelter, and any information as to special habits, for example, a man on nightwork or on fire-watching duties, etc.

To make certain that incidents are cleared, Incident Engineers have been appointed ; they will be responsible for certifying that such clearance has been effected. Experience has shown that no rescue job is hopeless : there must be perseverance on each and every job.

With Regional authorization, the following items have been added to rescue equipment :—

2 small car type jacks : 1-ton lift ;
2 hand trowels ;
1 pruning-saw (making two per party) ;
2 short-shafted round-nosed shovels ;
1 4-foot tommy-bar ;
1 slate-ladder.

Normally, heavy or bulky rescue tackle should not be taken to an incident. Special tackle of this type can be left in the Depots and sent for as required.

Demolition of Dangerous Buildings and Clearance of Streets.

In the air raids of December 1940, the demolition of dangerous buildings and the clearance of streets was carried out by the personnel of the City Engineer's department, with the assistance of the military and of one or two contractors.

Immediately daybreak arrived (about 9.0 a.m.), a senior assistant of the City Engineer's department and a senior Officer were allocated to each area.

Many roads were blocked, the damage being widespread over the city. A quick survey was made and the men set to work. Haulage and the necessary plant were provided and the work proceeded quickly.

Fire escapes proved very valuable in removing loose stonework from buildings, and petrol and petrol-electric cranes, Chaseside shovels, air-compressors, and excavators were put into use.

Demolition was carried out by means of explosives and pulling-down. Contractors, experienced in demolition work, also proved useful; they had practically no available labour, but were able to supply experienced men to take charge; they were also employed in burning-off steelwork.

Burnt-out tramcars left in the streets formed an awkward obstruction. They had to be lifted by cranes on to bogies and run in to depots.

Lessons.—On the morning after the heavier raid, Sheffield people tried to go to work as usual. This meant that they walked through the centre of the city, under buildings still on fire and dangerous. It is important that, at the earliest possible moment, a police cordon should be drawn around a blitzed area and that motor transport and pedestrian traffic should be diverted from the area.

Demolition by explosives is the only quick method, but discrimination is necessary to avoid extending the damage. Control must be exercised by the City Engineer. Many buildings, not reduced immediately, have since become dangerous. It would have been much cheaper to fell them at the time, by explosive.

Arrangements should be made in advance for the hire of cranes and other necessary plant.

Tips for debris should be arranged in advance.

Repairs to Roads.

The repairs to roads were carried out by the City Engineer, who took control of all work in bomb craters. The work was co-ordinated by a representative of the department, who contacted all service undertakings, including gas, water, electricity, and the Post Office. Where sewer repairs were concerned, these were at the greatest depth, and therefore took the longest time. Consequently, arrangements were made for temporary

service repairs to be carried out where necessary. The sewer was then repaired and the crater built up solid with hard core. Service repairs followed, and the road surface was reinstated temporarily. Main traffic and military routes were given preference, in both clearance and repair.

Road bridges were damaged, but in no case did repairs call for the use of Callender-Hamilton equipment. Specially-trained bridge squads carried out the repair of a damaged masonry arch bridge, where a heavy-calibre bomb had damaged the abutment, dislodged the springings of a portion of the arch, and cracked the barrel along lines radiating from the centre of the explosion. The method of repair was to pour in situ a reinforced-concrete relieving arch, 12 inches thick, below the existing masonry arch, and to re-set such of the masonry above this as could be recovered. Where the stones of the parapet and arch-ring were so shattered as to be unfit for re-use, new stone to match was obtained from a local quarry. The space left by shrinkage between the concrete and masonry arches was made solid by pressure grouting. For this purpose a grouting-machine was specially built up in the departmental workshops.

Lessons.—The value of a direct labour department was fully demonstrated in carrying out road repairs; a liaison officer, co-ordinating all service undertakings, proved extremely useful. Where services are in a footpath, a bomb crater in a carriageway can be repaired and the road opened to traffic, leaving the footpath railed-off for future repair.

The City Engineer should be solely responsible for filling in craters. In one instance, it was found that a crater had been filled in before the City Engineer's department had had an opportunity of discovering damage to the sewer.

Traffic-diversion is important. Adequate liaison and staff should be provided by the police, and traffic-diversion notice-boards should be supplied in advance.

Repairs to Sewers.

Damage to sewers occurred in many places, sewers varying in size from 6 inches to 48 inches diameter being affected, together with considerable lengths of rubble and brick culverts. Sewers were damaged to a depth of 16 feet to invert, and the damage often extended a considerable distance from the centre of the crater. Pipes cracked longitudinally at the springing level and crown. In one instance, pipes were cracked for a distance of 200 feet from the centre of the crater.

Before starting repairs, it was necessary to divert sewage to rivers and streams through existing storm-water overflows. Diversions of this type were carried out at different points. Penstocks were used, where available; otherwise, clay stanks were formed in manholes to divert the flow. These stanks were necessary in order to prevent flooding of craters, and to enable repairs to be carried out.

In all cases it was necessary to clear out the crater and ascertain the extent of the damage. This meant following the sewer along until the end of the damage was found. Generally, repairs were carried out with stoneware and concrete pipes. In a few instances, a double ring of brickwork 9 inches thick was reinstated. Where old rubble and brick culverts were damaged, concrete pipes were used with a taper closure in brickwork between the new pipes and the old culvert. In some instances the old invert was found to be in good condition and was left in, the sewer being made good by a brickwork arch or split pipes built up from the old invert.

Repairs were carried out mainly by direct labour of the City Engineer's department, but valuable assistance was also rendered by gangs from Bradford, Rotherham, the Sheffield sewage-disposal department, and a Public Works contractor who had been engaged on the repair of water-mains.

Portable power pumps proved very useful. Nine petrol-driven pumps, ranging from 2 inches to 4 inches in size, were available in the City Engineer's department, along with eleven air-driven pumps. Three other pumps were hired. The pumps were used mostly in pumping out flooded cellars and basement shelters and in dealing with sewage-flow in small sewers during repairs.

Air-compressors were used for breaking up concrete slabs in damaged roads, and, where repairs were being carried out at a considerable depth, portable cranes were used for lowering the larger-sized concrete pipes.

The repair of sewers was perhaps the biggest and most worrying of post-blitz duties. The shortage of trained sewer-men was very evident, and inexperienced labour could be employed only with very careful supervision.

Lessons.—Direct labour organization proved invaluable, as the men knew the conditions and routes of sewers. Key men with a knowledge of all overflows and penstocks on the sewerage system should be selected and allocated to different positions in the city. They act quickly on receiving instructions to screw down penstocks, or to form stanks in manholes for the purpose of diverting flows to rivers or along alternative sewers.

Portable pumps, up to 4 inches in size, proved very useful for pumping out flooded basement shelters and for pumping sewage from the manhole above the damage to the manhole below.

Damage to sewers usually extends for a considerable distance beyond the limits of the crater, and the sewer should be examined accordingly.

When damage is suspected in the vicinity of a crater, an inspection in the manwhole above the crater will often show if sewage is backing up, and will indicate if immediate repairs are necessary.

Efforts should be made forthwith to clear a crater sufficiently to permit a flow. This is particularly important if repairs cannot be carried out at once.

Adequate stocks of all sizes of stoneware and concrete pipes should be kept, the latter preferably in 4-foot lengths. In large-diameter concrete pipes, the ogee joint is preferred to the spigot-and-socket type. It is understood that Regional stocks of pipes, etc., are also available. Stocks of clay should be kept for stanking in manholes. Pits for the purpose of stocking the clay have been constructed in the Sheffield depots of the City Engineer's department. Manhole-covers should be examined and removed frequently, so as to enable speedy inspections when necessary.

Since the Sheffield blitz, a mutual assistance scheme has been arranged within the Region. This should prove extremely useful in providing experienced sewer-men and the necessary plant.

Repairs to Tramways Permanent Way.

Tramway-track was damaged in many places. Repairs had to follow sewer and service undertaking repairs, and therefore were delayed. Where no services were affected, temporary track repairs were completed rapidly. The exception was a crater, involving a 24-inch water-main and two important sewers: but an alternative track was available.

The length of track affected ranged from 20 to 40 yards, according to the size of the crater. The method of repair consisted of laying the tracks with new rails on wooden sleepers, ballasted in the same manner as a railway-track. The rails were fixed to the sleepers with dogs, the space between the rails being filled in with old bricks, and a temporary paving surface was laid. Permanent repairs were carried out during the summer, sleepers and ballast being removed and the normal concrete foundation and paving substituted.

In several instances, the effect of blast on the tramrails was rather curious. One rail was broken into small pieces and thrown a distance of more than 100 yards. Another piece of rail, 20 feet long, was thrown 20 yards and left standing vertically in an archway; 5 feet of the rail was buried in the ground, leaving 15 feet clear.

Lessons.—The temporary method of repairs by using sleeper track proved quick and successful. There was comparatively little subsidence. Advance organization of tram-track repairs is essential.

Final Conclusions.

Sheffield's experience showed the necessity of advance organization. There is a tendency to think that the best of organizations cannot stand up to the dislocation caused by a heavy blitz. This, to some extent, is correct, but the better the advance organization, the easier it is to build an improvised structure upon it. Improvization is necessary, but would be far more difficult without the solid foundation of well-considered advance organization.

Sheffield was complimented by the Regional Commissioner upon t
manner in which the services stood up to the heavy blitz air raids. T
was due to the service officers teaming well, and, in a great measure, to t
able leadership of the then Chairman of the Emergency Committ
(Alderman W. Asbury, J.P.).

Sheffield is much better prepared to deal with any future heavy a
raids. The services have been strengthened and an immense amount
training has been undertaken.

SYNOPSIS OF A PAPER FOR DISCUSSION.

The following Paper will be brought forward for discussion on the da
indicated in the margin of the synopsis, and will be published, with repo
of the oral and written discussions upon it, in the Journal. Memb
desiring to take part in the consideration of the Paper should apply fort
with for advance copies, which will be forwarded as soon as they a
ready. Applications for proofs should be made on postcards, quoti
the number of the Paper.

Date of Discussion 14/4/42.

"Post War Planning and Reconstruction."

By Herbert John Baptista Manzoni, C.B.E., M. Inst. C.E.

The Paper summarizes the present position of town and country pl
ning in Great Britain and contrasts it with the position in Europe and
United States. The recent acceptance of the necessity for natio
planning and reconstruction in Great Britain requires a completely n
organization, which is discussed in some detail in its development fr
basic policy, through the stages of national, regional, and detail plann
to a programme of construction and reconstruction on a fixed time schedu

The redevelopment of existing towns and the building of new towns g
opportunities for improvement of many existing services and
introduction of new ones.

Consideration is given to planning for defence in future wars, and
tasks which engineers must perform, both in planning and in reconstr
tion, are summarized, together with the additional factors which t
should study in order to take an appropriate place in the programme
work.

AERODROME ABSTRACTS.*

COMPILED BY THE DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH (ROAD RESEARCH LABORATORY) AND ISSUED IN COLLABORATION WITH THE AIR MINISTRY. THE ROAD RESEARCH LABORATORY AND THE MINISTRY OF WAR TRANSPORT JOINTLY COMPILE "ROAD ABSTRACTS" TO WHICH REFERENCE IS MADE.

1942, Vol. I, No. 1 (January). Abstracts Nos. 1–13

Note.—The abstracts purport to be fair summaries of the original literature, but no responsibility can be accepted by the Department of Scientific and Industrial Research for the accuracy of authors' statements or for their opinions.

Publication—Alternate months. A subject- and name-index will be issued annually.

1. **Airport Design Information**: U.S. Civil Aeronautics Authority: *Airport Division:* Washington, D.C., 1941 (U.S. Department of Commerce), $10\frac{1}{4}$ in. by 8 in., pp. 61, figs. 4, tables 6, unpriced. This manual discusses the fundamental principles involved in the selection of aerodrome sites, and in the design, planning, and construction of the aerodrome. The chapter headings are as follows: I. Factors to be considered in aerodrome planning; II. Factors influencing aerodrome size (includes tables giving standards for design, size and lighting); III. Factors influencing the selection of aerodrome site; IV. Investigation of soil conditions; V. Drainage system design; VI. Grading; VII. Aerodrome surfacings; VIII. Design data for paved runways; IX. Aerodrome buildings; X. The master plan. A bibliography is appended.

2. **The Highway, Street and Airport Manual**: Public Works Journal Corporation: New York, 1941 (Public Works Magazine), $11\frac{1}{2}$ in. by $8\frac{1}{2}$ in., pp. 122, numerous figs., unpriced; *Road Abstr.*, 1941, **8**, No. 338. This manual describes equipment, materials and construction methods used in building and maintaining roads and aerodromes. Section headings are as follows: Excavating and grading for highways and airports; drainage and sub-drainage for highways and airports; aggregates; data on bituminous materials for highways and airports; surface treatments; road-mix surfaces for highways and airports; plant-mixes for highways and airports; stabilization for highway and airport bases and surfaces; cement concrete for highways and airports; equipment and methods for highways and airport runway maintenance; and miscellaneous highway construction and maintenance equipment.

3. **Quartermaster Corps Instructions and Standards for Camps, Cantonments and Airfields**: U.S. War Department: *Office of the Quartermaster-General, Construction Division: Publ. Wks., N.Y.*, 1941, **72** (1), 33–4; *Road Abstr.*, 1941, **8**, No. 314. These instructions contain information and standards for the use of engineers and quartermasters. They include recommendations regarding water supply, sewer systems, drainage, and the construction of roads, runways and aprons. Standards are based on the U.S. Civil Engineering Standard Specifications. The following points are included: In general, *roads* shall have a maximum and minimum gradient of 1 in 12½ and 1 in 333 respectively. The surfacing should be the cheapest consistent with traffic and local conditions. Stress is laid on proper compaction of earthworks, the control of moisture content in placing filling, and adequate drainage. The gradient on *runways* is usually less than on roads. Where the gradient is altered, the change should not exceed 1 in 100. The steps in runway construction are similar to those in making roads of similar type. The cross-fall of aprons is generally 1 in 200 to 1 in 100. Accurate and detailed maps of lay-out, drainage, etc., shall be made and retained.

4. **Highway Principles and Practices applicable to Airports**: R. W. Crum: *Amer. Highw.*, 1941, **20** (1), 9, 14–16; *Rds and Bridges*, 1941, **79** (2), 21–2, 94, 98; *Road Abstr.*, 1941, **8**, No. 313. The requirements of road and aerodrome runway construction are compared and contrasted by the Director of the U.S. Highway Research Board, with special reference to the following considerations: (*a*) *Site*. In both cases a sufficiently large site should be secured to permit future expansion. Where the whole of the site is not to be used at once, a carefully prepared plan of stage construction should be followed. In choosing both aerodrome and road sites it is most important to make soil surveys, to ensure adequate drainage and stability of the sub-grade in all weather conditions. (*b*) *Road and runway foundations*. Soil surveys indicate whether the subsoil is likely to require more than surface drainage, and whether the existing soil will provide a satisfactory base course without stabilization. Stabilization is necessary if the soil is liable to volume changes when the moisture content varies. The methods generally used for roads are also applicable to runways and landing strips; they include the blade-mix, travelling plant-mix, and stationary plant-mix methods. In general, the stationary plant-mix method is recommended for mechanically stabilized base courses, while mixing for bituminous stabilization is generally done in a travelling plant or by blading. Adequate compaction and "seasoning" are even more necessary in constructing runways than roads, since consolidation under traffic is practically negligible on runways. (*c*) *Surfacing*. The basic principles of design, to provide adequate bearing power and resistance to stresses imposed by traffic, are the same for both road and runway surfacings, but their application will differ inasmuch as the traffic frequency

on runways is much less than on roads. Formulas developed for the design of flexible road surfacings from 1901 onwards are reviewed; they include the work of B. E. GRAY (see *Road Abstr.*, 1935, **2**, No. 262), A. C. BENKELMAN (see *Road Abstr.*, 1938, **5**, No. 137), and P. HUBBARD and F. C. FIELD. As a working measure, the last two authors define the load-supporting value of the soil as the load in lb./sq. in. that would cause a deflection of ½ in. on a circular disc of surfacing equal in area to the area of contact of a lorry-tire designed for a 12,000-lb. load. Hence on a given subgrade the thickness of bituminous surfacing required is that which will just deflect ½ in. under the design load. The absence of the consolidating effect of traffic on runways makes advisable the use of softer binder, richer mixes, and more densely graded aggregates than for road surfacings of similar type. In addition, runway surfacings should possess a fine, close texture, since coarse-textured surfacings cause undue wear on tires. (*d*) *Dynamic loading.* The severe dynamic stresses caused by landing aircraft constitute a special problem in runway construction. As a result of tests made to determine the effect of these impact stresses on the aircraft, the runway, and the tires, an "impact factor" of 1·25 to 1·5 has been recommended, to be applied to the unit load on the surfacing. Further research is necessary regarding the distribution of the impact loads to the subgrade, for, although with heavy aircraft these loads may be very large, their duration is brief owing to the speed of the aircraft. Another problem requiring further study is the factors affecting the durability of surfacings. Co-operation between engineers engaged in road and aerodrome construction is recommended to obtain solutions of these problems.

5. **Landing Field Design**: M. J. ADAMS: *Bull. Soc. Am. milit. Engrs*, 1941, No. 5, 7–11. The runway type of aerodrome is compared with the all-over type for military purposes. Runway intersections are vulnerable to bombing and the capacity of a runway is limited to one plane landing or taking-off at a time. It is suggested that where soil conditions are suitable some all-over type landing fields should be constructed. By control of the ground-water level heavy loads can often be catered for without the use of surfacing. The United Airport (Rentschler-Field) at East Hartford, Connecticut, is an example of the all-over type that is giving good service after 10 years' use. The only maintenance cost is cutting grass and removing snow. The drainage system was cheaper than for the runway type design. A bituminous coated, perforated, corrugated metal pipe was used because of its ability to withstand impact loads when installed near the surface. To prevent destruction of the turf and avoid dust nuisance, concrete aprons were built near the hangars and from these bituminous macadam strips lead to taxi strips of the same material.

6. **Airport Drainage and Sub-Drainage**: ANON.: *Publ. Wks., N.Y.*, 1940, **71** (11), 13, 44–5; *Road Abstr.*, 1941, **8**, No. 286. This review of the principles of airport drainage is based on a Manual of Airport Drainage

issued by the AMERICAN ROLLING MILL CORPORATION. The requirements, though fundamentally the same as in highway construction, show notable differences in detail. Aerodrome drains must possess a high resistance to impact. Additions and repairs to an existing system cannot readily be made after the runways are complete, and design should therefore be based on an estimate of maximum future requirements. While in both cases surface water must be removed promptly, ditches and channels on an aerodrome constitute sources of danger. In many cases a single system suffices for surface and subgrade drainage. On small aerodromes with unsurfaced runways a relatively small amount of surface drainage will be needed ; if the runways are surfaced the entire drainage system must be based on their requirements. Soil surveys should be made to provide information regarding the character of the soil, the stability of the subgrade, and the position of the ground-water level and of seepage areas. The depth and spacing of the drains depend on the nature of the soil ; the usual depth is 2 to 4 ft., and the spacing varies from 25 to 30 ft. in clay to 100 ft. or more in highly permeable soils. Spacings are tabulated for 10 types of soil containing stated proportions of sand, silt and clay, and the method of installing perforated metal pipes is illustrated for grass areas and for the margins of surfaced runways (see also *Road Abstr.*, 1937, **4**, No. 206). The required capacity in cu. ft. per sec. per acre and per square mile is tabulated for drains designed to remove $\frac{1}{16}$ in. to 1 in. depth of water in 24 hours. The cross-section of runways must have sufficient camber to permit rapid surface drainage and the surface water should be intercepted by piping. The pipe is usually entirely surrounded by the permeable course, and is laid with the perforations downwards to prevent the entry of silt. If it is necessary to prevent the escape of water before the outlet is reached, the pipe is laid with the perforations upwards on impermeable material which surrounds all but the upper third of the circumference. It is recommended that the basis for calculating the capacity of the surface drainage system should be an hourly rainfall of an intensity exceeded not more than once a year ; the system should be capable of removing the corresponding volume of water within 2 hours after rain ceases. Hourly rates recommended for various regions of the U.S.A. and found satisfactory over 14 years are 1·8 in. for the Gulf States, 1·4 in. for other South-Eastern States, and 1 in. for the remaining States east of the Rocky Mountains.

7. **Airport Drainage is important:** W. A. MASON : *Bull. Soc. Am. milit. Engrs*, 1940, No. 3, 17–9. The drainage of runway-type aerodromes is discussed. With surface-oiled or paved runways about 90 per cent. of the rainfall flows to the surface drainage system. These runways need comparatively large trenches for drain pipes to remove the surface run-off before it becomes dangerous for landing aircraft. The determination of sizes of drains depends on probable maximum rainfalls, available outlets

and gradients, size of runways, and time allotted to remove the surface water. Because of the great length of runways required by heavy aircraft, the drainage system must usually be laid on relatively flat gradients. The surface run-off is generally collected by continuous interception by drains placed alongside the runways. Crushed rock or gravel backfill in the drain trenches passes the water directly to the drains and hence to the outlets. It is necessary to maintain the surface of these trenches against displacement by taxi-ing planes, or by the suction from airscrews. The runways are crowned slightly to direct the surface flow to the side trenches, in which the surface of the backfill is flush with the edge of the runways. The alternative plan for surface water interception is to provide surface inlets equipped with gratings at frequent intervals along the sides of the runways. The objections are the excessive concentration of surface water around the inlets after heavy rain and the introduction of inlet structures of rigid materials near the runways with consequent danger to landing aircraft. To avoid breakage and disalignment from impact blows, flexible metal drainpipes should be used. Examples of aerodromes using such pipes are given.

8. **Stormwater Drainage in Flat Country**: E. D. Grubb : *Surveyor*, 1941, 99 (2576), 359–60. In flat districts where the subsoil is impermeable and artificial drainage canals are necessary to prevent flooding, the drainage of aerodrome sites must be more efficient than is needed for agricultural land. A method suggested is to provide agricultural pipes at intervals of about 20 yds., to place tubes in ditches which are filled-in, to isolate the aerodrome site from the remainder of the drainage area, and to provide a dyke round the perimeter from which water can be pumped at times when the land drainage dykes are not overloaded. The drainage system of a 200-acre aerodrome in a low-lying district is given as an illustration. The dyke round the perimeter is 6 ft. deep and 9 ft. wide and the length of agricultural pipes at 20-yd. intervals is 750 ft. per acre. The pipes that take the place of ditches should be of a cross-sectional area not less than that of the water in the ditches in wet weather. For this example 20,000 ft. of 18-in. pipes is required.

9. **Mole Drainage in New Zealand**: A. W. Hudson and H. G. Hopewell : *Department of Scientific and Industrial Research, New Zealand, Bulletin* No. 86. Wellington, 1941 (The Department), 9¾ in. by 6 in., pp. 91, ill., 2*s*. 6*d*. The comprehensive information on mole drainage contained in this publication includes descriptions of the equipment used, mode of action and life of mole drains, preparation of the ground, systems of mole drainage, methods of connecting mole to tile drains, and cost. The method is suitable for draining large areas where the subsoil contains a relatively high proportion of close, compact clay, almost devoid of organic matter. Mole drains are usually drawn between 14 and 30 in. below the surface.

10. **Experience of Vertical Surface Water Drainage on Aerodromes:** BANNERT: *Flughafen*, 1941, 9 (1/2), 12–4. The care of grass-covered aerodromes is discussed with special reference to the question of drainage, particularly in heavy soils. The upper layer of soil must be rendered sufficiently permeable to allow the water to be quickly absorbed and/or led off to the lower strata. Adequate moisture must, however, be retained to encourage plant growth. *Mechanical methods* of dealing with water-logged soil include providing drainage systems; removing strata of conglomerate, formations of bog iron ore, or local rock, by blasting; cutting drainage channels, planting taproots, and breaking up compacted soil with a "goose-foot" machine (a kind of rotary scarifier). *Chemical methods* include treating soils containing excess sodium with gypsum; using acid fertilizers for alkaline soils, and vice versa.

11. **Design of Airport Runways:** U.S. WAR DEPARTMENT: *Office of the Chief of Engineers:* Washington, D.C., 1941 (U.S. War Department, Corps of Engineers), 10½ in. by 8 in., pp. 195, ill., unpriced. This manual indicates the principal requirements of aerodrome runways, and suggests criteria and methods of design. For the principles underlying the layout and dimensions of runways, the reader is referred to the current standards of the U.S. CIVIL AERONAUTICS AUTHORITY (see Abstract No. 1). Attention is chiefly paid to the types of surfacing most suitable for runways and the basis of selection, the principles of design of surfacings and drainage systems, suitable materials and specifications for construction, general construction methods with particular regard to design, and examples of construction of surfacings and drainage systems at aerodromes in the U.S.A. The subject matter is treated under the following main heads: general features of design; concrete surfacings; bituminous surfacings; soil stabilization; and drainage. Four appendices contain, respectively, a comprehensive list of references, data for constructing concrete runways (including aircraft tire sizes, wheel loads, contact areas, subgrade bearing capacities, etc.) and bituminous runways, and examples of runway drainage.

12. **Asphalt-Stone Construction for Airport Surfaces:** A. H. HINKLE: *Crush. Stone J.*, 1941, 16 (2), 9–12, 17–8. Asphaltic surfacings are claimed to fulfil the following requirements for aerodrome runways, namely, that the surface should be smooth, strong and durable, non-skid, free from loose particles, of an appropriate colour, and of such a character that it can be repaired with ease and speed. Surfacings suitable for heavy-duty runways are discussed, including road-mix penetration macadam and hot plant-mix asphaltic concrete. Examples of aerodrome construction are described, including the airport at Morgantown, West Virginia (see following abstract). The Cleveland Municipal Airport, which carries very heavy traffic, consists of a 5-in. water-bound slag base, 2-in. asphaltic penetration macadam binder course, and 1½-in. cold-laid asphaltic concrete

of the amiesite type as a wearing surface. This was given a seal coat. The total thickness was only 8½ in., on a somewhat unstable soil, but if the surface became rough through distortion it could be repaired with 1½ in. to 2 in. of new asphaltic concrete of hot-mix type.

13. **Bituminous Concrete of Crushed Stone Screenings and Asphalt used in Wear-Course Construction, Morgantown, West Virginia, Municipal Airport**: A. M. MILLER: *Crush. Stone J.*, 1941, 16 (2), 13–6. The municipal aerodrome at Morgantown, West Virginia, has 3 runways, 3,600, 3,200 and 3,100 ft. long. The drainage facilities were good as the aerodrome is situated on a high, narrow ridge. The total excavation required in levelling the surface for the runways was approximately 913,000 cu. yd. of which 500,000 cu. yd. was sandstone. The base course for the runways was obtained from the excavated material and was constructed of knapped stone placed in two 6-in. layers. Each layer was rolled with a 10-ton, three-wheeled roller and the top layer blinded with limestone screenings—grading ⅜ in. to dust. Type and thickness of surfacing were influenced by: limited funds for materials; the use of a minimum amount of equipment and the maximum amount of labour; and the local availability at low cost of limestone aggregates ⅜ in. to dust and 1 in. to 4 mesh. Laboratory experiments carried out by the WEST VIRGINIA STATE ROAD COMMISSION led to the adoption of a two-course pre-mix surfacing, using rapid-curing cut-back bitumens. For two of the runways 31 cu. ft. of stone and 11·5 gal. of RC-3 were mixed in each batch of binder course; mixing time ½ min.: 21½ cu. ft. of stone and 24·5 gal. of RC-2 were mixed in each batch of wearing course; mixing time 1½ min. On the whole job the maximum day's output for binder course was 400 tons and for wearing course 250 tons. The mix was machine-spread and rolled with a 10-ton, three-wheeled roller to give compacted thicknesses of 1½-in. binder and ⅝-in. wearing course. Plant and construction details and cost data are given. The first runway was completed in 1937 and has been used since by heavy aircraft and the contractor's heavy lorries: its present condition is highly satisfactory.

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH.

Timber Economy in Building.

Less timber must be used in building. The object of Wartime Building Bulletin No. 19 (Timber Economy in Building) * is to show how this can be achieved. The general approach to the problem is given under five headings :—

(1) Omission of all unessential features.
(2) Use of alternative materials.
(3) Better use of material and economical design.
(4) Designing so that stock sizes can be utilized.
(5) Use of substitute types of timber in place of those normally used for particular purposes.

Various uses of timber in which considerable economies could be affected are listed and a section is devoted to each main use.

Some of the suggested recommendations could be applied without any difficulty. In other cases there would have to be a proper appreciation of the problems by the designer and collaboration by the manufacturer or contractor. A number of illustrations show designs for benches, shelves, shutters, joinery fittings, doors, storage bins, etc.

Apart from many detailed items of interest the chief points which arise are :—

(1) The need for reviewing all designs to see how timber content can be reduced. This involves the examination of actual requirements based on strength and other factors. Information now available on timber grading makes it possible to calculate strengths much more accurately than has been customary and should lead to a revision of ideas previously based solely on tradition.

(2) The important part which plywood can play in modern building. Under present conditions very considerable economy can be obtained by substituting plywood for solid timber.

(3) For temporary timber work, such as shuttering for concrete, standardization is important so that a maximum re-use can be obtained with the minimum of alteration to the shuttering.

(4) All designs should be related to available stock sizes. An appendix gives details of the stock sizes of plywood and solid timber available at the present time.

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* Copies of Wartime Building Bulletin No. 19. "Economy of Timber in Building", issued by the Building Research Station of the Department of Scientific and Industrial Research, may be obtained from H.M. Stationery Office, price 1*s.* net.